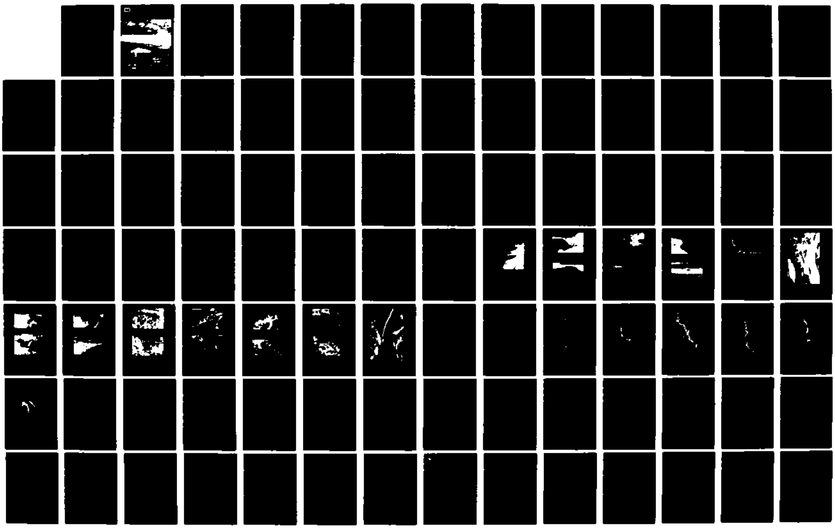
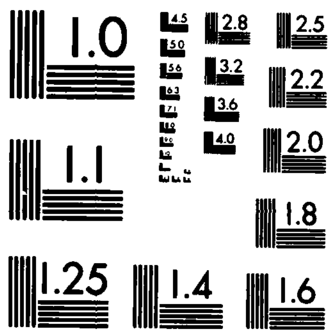
AD-A121 138 THE STREAMBANK EROSION CONTROL EVALUATION AND 1/4
DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER
WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..
UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

(12)

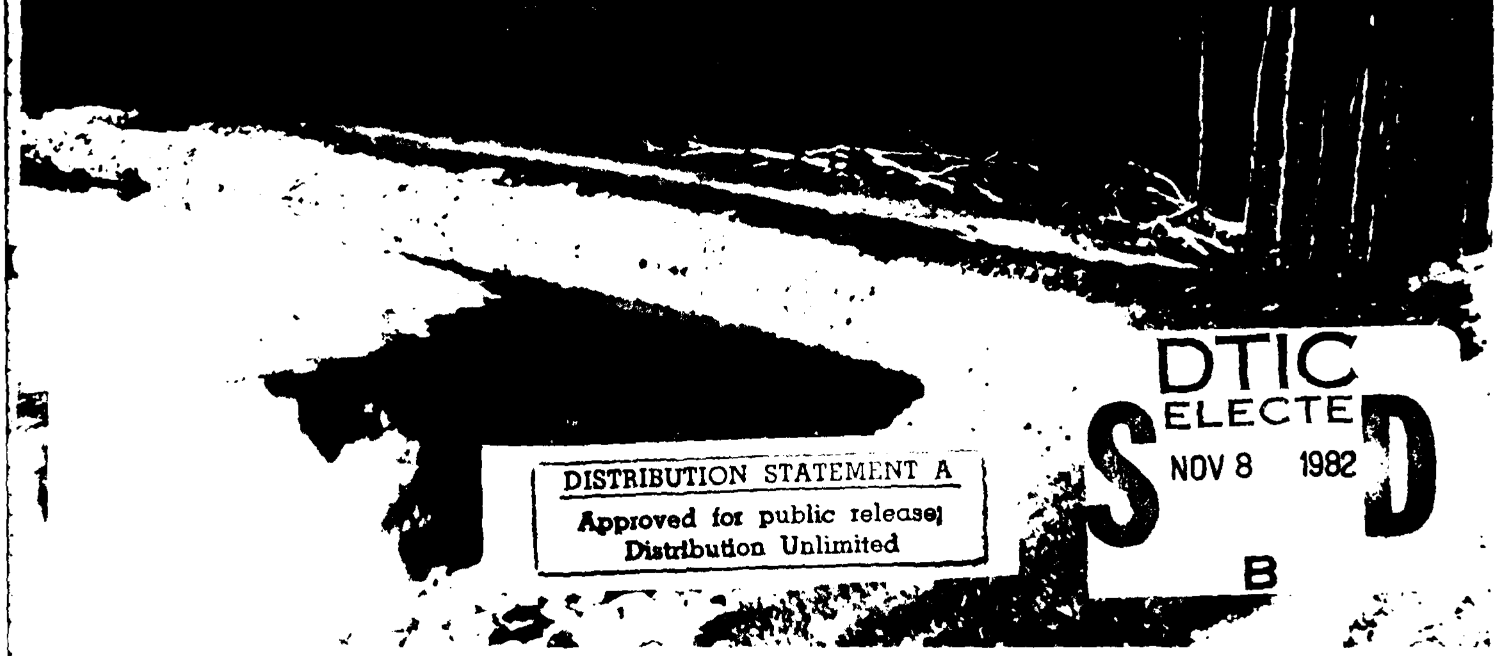
US Army Corps
of Engineers

December 1981

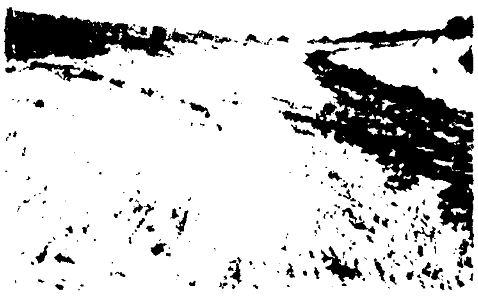
**THE STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION ACT OF 1974
SECTION 32, PUBLIC LAW 93-251**

Appendix G - Demonstration Projects on Other Streams, Nationwide

Volume 1 of 2



Rock Toe With Tie-Backs



Precast Block Paving

Board Fence Dikes

FINAL REPORT TO CONGRESS

THE STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION ACT OF 1974
SECTION 32, PUBLIC LAW 93-251

APPENDIX G

DEMONSTRATION PROJECTS ON OTHER STREAMS, NATIONWIDE

VOLUME 1 OF 2

Consisting of

A BRIEF SUMMARY REPORT AND INDIVIDUAL EVALUATION
REPORTS ON TWENTY STREAMBANK EROSION CONTROL
DEMONSTRATION PROJECTS ON SIXTEEN DIFFERENT STREAMS
THROUGHOUT THE UNITED STATES

U.S. ARMY CORPS OF ENGINEERS
December 1981

APPENDIX G

Demonstration Projects on Other Streams, Nationwide

CONTENTS

VOLUME 1 of 2

VOLUME 2 of 2

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

FINAL REPORT TO CONGRESS

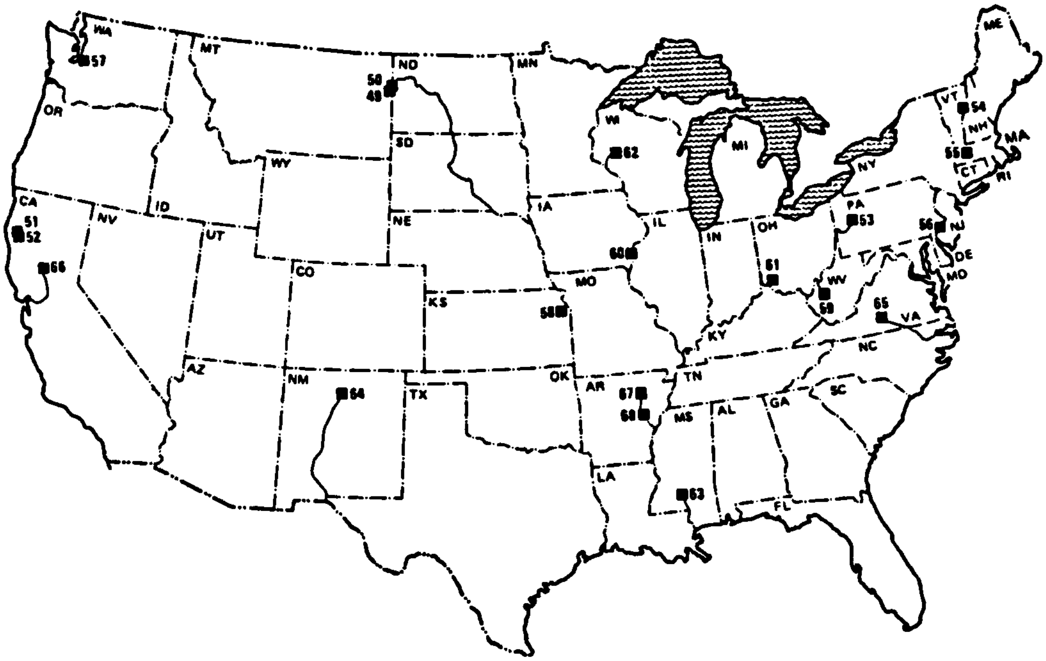
APPENDIX G

Demonstration Projects on Other Streams, Nationwide
(Work Unit 8)

Summary

1. General. This appendix presents the results of demonstration projects constructed nationwide to reflect a variety of geographical and environmental conditions, including naturally occurring erosion as well as erosion caused or increased by man's activities. Also included are projects constructed in response to amending legislation on the lower Yellowstone River and in the delta of the Eel River. Figure 1 presents the location of the projects. Detailed reports for each of the twenty projects are included in this appendix, in the order given in Figure 1.

2. Site Conditions. While there were similarities in the geotechnical and hydraulic conditions, the sites are in reality as widely varied as their geographical spread. The riverbanks all consist of erodable, coarse (sands and gravels) to fine (silts and clays) grained materials. These materials had been deposited by alluvial and eolian processes in geologically recent periods. At the various sites, the actual materials and their arrangement varied considerably. At some locations the various sizes of materials were heterogeneous for the full height of the exposed bank while at other locations the materials were stratified with interbedded layers of differing materials. Some banks were free draining while others were very slow draining. All eroding and failing banks, which varied from 3 to 100 feet in height, were very steep. Design flows varied from zero (where waves were the cause of the erosion) to about 200,000 cubic feet per second (cfs), while the flood potential at the various sites varied up to a maximum of over 500,000 cfs. Flow velocities varied from zero to a maximum of about 14 feet per second (fps), while river slopes varied from flat (upstream of dams) to about 11 feet per mile. At one location, the



YELLOWSTONE RIVER (Intake, Montana to Mouth)

49. River Road, Montana (Near Sidney, Montana)
50. Cheney Creek, North Dakota (Near Cartwright, North Dakota)

EEL RIVER DELTA

51. Eel River, Fernbridge, California
52. Van Duzen River, Carlotta, California

PROJECTS ON OTHER STREAMS, NATIONWIDE

53. Allegheny River, Wattersonville, Pennsylvania
54. Connecticut River, Haverhill, New Hampshire
55. Connecticut River, Northfield, Maine
56. Delaware River, Paulsboro, New Jersey
57. Green River, Kent, Washington
58. Kansas River, Eudora, Kansas
59. Kanawha River, South Charleston, West Virginia
60. Iowa River, Wapello, Iowa
61. Little Miami River, Milford, Ohio
62. Lower Chippewa River, Eau Claire, Wisconsin (5 sites)
63. Pearl River, Monticello, Mississippi (3 sites)
64. Rio Chama, Abiquiu, New Mexico
65. Roanoke River, Leesville, Virginia (2 sites)
66. Sacramento River, Ordbend, California
67. White River, Jacksonport, Arkansas
68. White River, Des Arc, Arkansas

Figure 1. Locations of Demonstration Projects on Other Streams, Nationwide

water surface is always under the influence of ocean tides and at another location, tides affect the water surface only during periods of low to moderate flood flows. Water surface levels were affected by dams at eight sites (four located upstream of dams, two affected by releases from flood control dams and two affected by releases from hydroelectric power plants). Floating debris or ice contribute to the problem at twelve sites.

3. Instability and Erosion Problems. At all locations the instability and/or erosion problem is the result of a combination of many actions or causes. Usually two, three, or four causes are predominant and the others can be ignored. Only occasionally is the problem the result of a single action or cause. Consequently, it is practically impossible to positively state the cause(s) or to list the possible causes in order of influence. As each site is unique, the conditions existing at each location must be determined and then evaluated to establish the probable cause(s) of bank instability or erosion. As a minimum, one must have some information on: what materials are in the bank, how they are situated, and the effect of groundwater drainage on physical properties of the bank; the velocity of flowing water acting on the bank; variation in water surface levels, rate of change in level, and whether as a result of natural or other causes; the possibility of ice, floating debris, or mineral debris affecting the bank or flow conditions along the bank; and stream geometry (both in plan and cross section) and how it changes as a result of flood flows and with time. The primary causes of bank recession acting at the sites reported on in this appendix are briefly discussed in the following subparagraphs.

a. Tractive Erosion. The shearing force exerted by fast flowing water is the most prevalent cause of bank erosion. It probably causes more erosion than all other causes combined. Tractive erosion usually results when channel irregularities (meanders), snags, landslides, etc., cause the higher velocities, normally found in the central portions of a stream, to impact on or to flow along the bank. The usual pattern of erosion is for the high velocity to erode the toe or bottom of the bank, removing support of the upper part of the bank which then slumps

into the stream and the cycle repeats itself. For this reason, the protection afforded the toe of the riverbank is usually the most critical part of any bank protection scheme. Successful riprap revetments usually result when available, empirical techniques are followed, but little information is available as to why the techniques work or how they may be modified to be less destructive of the environment yet still provide the desired protection to the riverbank. Even less is known about the use of other materials or methods that might be used to protect the bank. As might be expected (considering its prevalence), tractive erosion was identified as significant, if not the most significant, cause of erosion at 17 of the 20 projects discussed in this appendix. The three projects where this cause was not identified were those on the Delaware, Kanawha, and Roanoke Rivers.

b. Ice and Debris. Floating ice and debris (trees, lumber, etc.) moving with the current, the gouging action of ice jams, and the lifting and crushing action of ice rising and falling with changes in water level were found to be a significant cause of erosion and a cause for damage to bank protection works. Ice problems are confined to areas that experience colder climate, whereas floating debris problems are prevalent on medium to high velocity streams (particularly those flowing through steep, heavily wooded terrain). Significant ice problems were identified at both projects on the Yellowstone River and the projects on the Allegheny, Iowa, Kanawha, and Lower Chippewa Rivers. Ice action, though not of major significance, was also identified at the Little Miami River and both Connecticut River projects. Floating debris problems occurred on the Eel, Van Duzen, Iowa, and Green Rivers.

c. Waves. The impact and wash action of waves was the cause of erosion at the Delaware River Project.

d. Overland Flow. Surface runoff from upland areas flowing over the top of the riverbank causes gullies that degrade the bank and weaken the bank so it is more vulnerable to other erosive actions. The erosive action of overland flow was identified at the Green and Pearl River Projects.

e. Long Duration Reservoir Releases. Releases from flood control reservoirs frequently result in long periods of bank-full, or nearly bank-full, flows, whereas the natural floods would have significantly exceeded bank-full capacity, but for a relatively short period of time. Reservoirs therefore expose the full height of the riverbank to high-velocity, eroding flows for longer than natural periods. As a consequence, a riverbank that was protected by native plants or that was slowly eroding under natural flood flow conditions, can become a significant erosion problem as a result of reservoir releases. The erosion protection features on the Kansas River and the Rio Chama had to address such problems.

f. Geotechnical Caused Instability. Normally, the only geotechnical consideration given to the design of bank protection is an evaluation of the material exposed on the riverbank and whether or not a filter will be required under the primary protection to prevent the loss of individual soil particles. The Section 32 Program included a more substantial subsurface evaluation of the in-place bank materials to determine, if possible, how riverbanks erode, if there was any relationship between soil type and extent of erosion, and if there were any soil characteristics that contributed to erosion. As a result of these investigations, it was learned that there are characteristics which contribute to erosion, and some which are a major cause of erosion. River rises, whether as a result of flood flows or hydroelectric powerhouse operations cause the bank to become saturated. Then on a falling river stage, depending on what materials were in the bank and how they were mixed or placed in layers, any one of four forms of bank instability could result. On the Lower Chippewa River, it was found that following saturation of the bank as a result of upstream power plant operations, the return seepage was causing surface sloughing within the first few inches of the soil surface. On the Pearl River after high flood stages and on the Roanoke River as a result of upstream power plant operation, water pressure in the riverbank caused the bank to slump. Internal pressures from flood flows caused block failures on the Allegheny River. At Des Arc on the White River, drying

of the clays as a result of flood flows and rainfall caused cracks to open behind the bluff face. Subsequent high water and rainfall would induce internal pressures (which would further reduce the soil's shear strength) and would soften the underlying, highly-plastic clays, which would then permit large blocks of the bluff to collapse. Groundwater flowing to a river in a layer of cohesionless soil (usually sands and silty sands) frequently carries soil particles with it. This gradual mining of particles out of a layer eventually over-stresses overlying layers causing them to slump into the river. This action, termed "piping," was found to occur on the Allegheny and Kanawha Rivers following flood flows and on the Green River after intense local rainfall.

4. Protection Methods and Materials. The Section 32 Program Steering Committee provided general guidance on how to select and develop a demonstration project site (generally a site should be 1,000 to 2,000 feet long, evaluate at least three methods and/or materials, and land protected should not be owned by the Federal Government, etc.). Designers were given almost complete freedom to select materials to be used and how they should be placed on the bank. Designers were required to present a rationale for their choices and assumptions, and show why the proposed materials and methods were expected to protect the riverbank from all flows up to the design flood. In almost all cases the design flood was less than would normally be used at a Corps of Engineers project, i.e., the 100-year or standard project flood. In the following subparagraphs, the materials used along with the method of placement and where used are briefly presented. Many of the protection methods included vegetation as a significant part of the protection plan. But, with one exception, all of these plants relied on some other material to protect the toe, hold the vegetation in place, and/or preclude erosion around the plants. All the vegetation plans (other than the one exception) have been listed under the other materials used rather than under vegetation.

a. Methods Primarily Involving Rock. Rock is, in most cases, the most readily available and most durable material that can be used for bank protection. It is also frequently the most (or nearly so)

economical, particularly when procurement and placement will be accomplished under a construction contract. In most cases where rock was used in this program (and as described below), the rock was used along the toe of the riverbank where other materials are difficult, if not impossible, to place. In a few instances, conventionally designed riprap revetments were included in the demonstration projects (riprap on filter cloth - Delaware River, stone paving on graded and ungraded slope - White River (Jacksonport), riprap - Allegheny River, Little Miami River, and Rio Chama). These riprap revetments were included in the projects to prevent the untried protection methods from being flanked and/or to form a basis for comparing the effectiveness and cost of the untried methods. Other methods making significant useage of rock follow:

(1) Rock Toe. At several locations conventional riprap was placed underwater at the toe of the riverbank, while the upper part of the bank was protected with other materials. On the Allegheny and Delaware Rivers, such a toe was placed in front of existing wood bulkheads. Except for the planting of some willows, the upper bank was left in its natural condition on the Eel River. The upper slope was graded to a uniform slope and covered with alternating strips (up and down the bank) of clay and rock on the White River at Jacksonport. Two extensive planting schemes were placed on the graded upper slope at the Sacramento River Project. Several planting schemes were tried on the Green River above the rock toe.

(2) Rockfill and Stone Dike at Toe. A rock-filled toe generally consists of placing a large rock fill against the existing bank near the toe, instead of preparing the slope and placing a conventional riprap layer and toe detail. Rockfills of this type were used at the Allegheny River, Kanawha River, and Chippewa River (3 variations) Projects. At Des Arc on the White River, a trench excavated at the base of the eroding bluff was backfilled with rock to stabilize the bluff. A stone dike is a free standing rock fill placed at and parallel to the toe of the riverbank. The area landward of the dike is then filled to provide

the desired upper bank slope. Stone dikes were constructed on the Allehgeny, Roanoke, and Yellowstone (at Cartwright) Rivers. The projects on the Kanawha and Roanoke Rivers had vegetation planted on the upper bank, and on the Yellowstone River, narrow strips of stone alternating with slightly buried willow facines were placed at intervals on (extending up and down) the upper bank with vegetation planted between the strips.

(3) Windrow Revetment. In a windrow revetment, the rock is placed on and near the edge of the top of bank. As the bank continues to erode, the rock will be undercut and will be self-launched down the riverbank to form a rock revetment. On the Allegheny and Roanoke Rivers, the rock was placed in an excavated trench and then covered with soil, while on the Kansas River the stone was piled on top of the ground. At the Kansas River Project, different weights of stone (tons per foot) were placed to try to determine the optimum quantity.

(4) Rock Hardpoints. Hardpoints are rock fills placed at intervals on and/or into the riverbank to keep the higher velocities, ice, debris, etc., away from the bank. While some material is expected to be eroded between the hardpoints producing a scalloped effect on the bank, the amount of erosion will vary depending on the spacing of the hardpoints and the amount, type, and size of vegetation growing between the hardpoints. Rock hardpoints were constructed on the Allegheny, Eel, and Chippewa (2 variations) Rivers.

(5) Rock Reinforcement. On the Green River, quarry spalls were mixed with native backfill which was then placed in a two-foot layer on the upper bank. Several vegetation schemes were then planted on the bank.

b. Methods Using Manufactured Products. The protection methods presented in the following subparagraphs utilize standard items, which, in almost all cases, are manufactured specifically for bank protection.

(1) Gabions. Gabions are totally-enclosed, wire baskets that are filled with rock. The individual baskets are wired together to form an integrated structure. At Milford on the Little Miami River, a gabion wall was constructed at the toe of a high, steep bank. Fill was placed behind the wall and vegetation was to be placed on the fill. Groins (dikes) were constructed in the Rio Chama using gabions. On the Delaware River and the Connecticut River at Haverhill, a gabion mattress (revetment) underlaid with filter fabric was placed on the bank.

(2) Concrete Blocks. Precast concrete blocks with filter fabric underneath were used on the Pearl, Chippewa, and Connecticut (at Northfield) Rivers. On the Chippewa and Pearl Rivers, a rock toe was used with blocks, while on the Connecticut River, the blocks were used for toe protection, and vegetation was placed above the normal water level.

(3) Various Types of Matting. Wire mesh was used on the upper bank of the Green River to reinforce the soil surface while plantings become established. Two types of grout-filled fabric mats, Fabriform and VSL Hydro-Lining (two thicknesses of nylon cloth attached to one another to obtain a uniform thickness when concrete grout is injected between the two cloths), were used on the Iowa and White Rivers, respectively. Four types of mats (with variations) were used on the Chippewa River - two types of Enkamat (an open, plastic-fiber mat), continuously anchored along top and bottom edges and randomly in between, and two arrangements with snow fencing laid on the bank.

(4) Kellner Jacks. Kellner Jacks (constructed of crossed, steel angles and wire) were placed in a line parallel to the bank of the Iowa River.

(5) Fencing. Woven wire fencing, with cut brush fill landward of the fencing, was used on the Chippewa River. Two densities of timber piles and bracing were placed in combination with woven wire fencing on both the Eel and Van Duzen Rivers. A standard, riprap toe was placed in front of the pile fences at the Eel River site.

(6) Concrete Cribbing. A concrete crib wall was placed at the toe of a high steep bank on the Little Miami River. Vegetation was placed on the backfill behind the wall.

(7) Longard Tube. A Longard Tube (a large neoprene-fabric tube filled with sand) was placed along the bank of the Delaware. Two types of protective covers for the tube were also installed.

(8) Concrete Piles. Concrete piles were cast in place in the bank of the Sacramento River to form embedded groins (dikes). These embedded groins will not function until there is additional bank erosion. Then they will function in the same manner as the rock hardpoints discussed above.

c. Methods Using Available Materials. The following subparagraphs present bank protection methods where commonly available materials were used.

(1) Sand Dike. A sand dike was constructed across a secondary channel on the Yellowstone River (Sidney). Various vegetation plans were planted in strips across the dike.

(2) Bagged Revetments. Sand filled bags were used on the Chippewa and Delaware Rivers. On the Chippewa River, non-biodegradable bags were utilized, while nylon bags were utilized on the Delaware River. Reinforced paper bags filled with a sand-cement mixture were placed on filter fabric on the Connecticut River at Haverhill. Bagged soil-cement was used on the lower bank and vegetation on the upper bank of the Kanawha River.

(3) Soil-Cement. Cast in place soil-cement was used on the unstable bluff at Des Arc on the White River.

(4) Wood, Including Trees and Logs. Permeable timber jetties (dikes) were used on the Iowa River. Log (telephone pole) cribs were

constructed on the Rio Chama. Tree trunks (pendants) were placed so they overlapped one another on the Van Duzen River, while on the Chippewa River trees were anchored to the bank at various spacings.

(5) Used Tires. Mats constructed of used tires were constructed at five locations - the Pearl River, the White River at Des Arc, the Kanawha River, the Roanoke River, and the Connecticut River at Haverhill (both with and without filter fabric under). Vegetation was planted above the mats on the Kanwaha and Roanoke Rivers. Walls or bulkheads were constructed on the Delaware River where the bulkhead was constructed both with and without toe protection and on the Connecticut River (at Northfield) where the bulkhead was in part backed by filter fabric and where vegetation was placed above the bulkhead. A floating tire breakwater was constructed on the Kanawha River.

(6) Rubble. Dumped rubble was placed on the bank of the Pearl River.

(7) Baled Hay. Baled hay was used to protect the lower bank of the Connecticut River at Haverhill. Vegetation was placed on the upper bank.

d. Vegetation. Vegetation alone was placed on the bank at Haverhill on the Connecticut River. However, the toe failed taking with it some vegetation. Toe protection was added and the vegetation replaced.

5. Maintenance and Rehabilitation. Some of the erosion protective systems used under this program are low cost schemes which are designed to protect the bank against a limited number of critical events, or just mitigate the damages of a specific event. As a result, a higher level of maintenance will be required throughout the life of the structure. If the structures are permitted to degrade, the eroded bank condition will redevelop. The cost of the maintenance and rehabilitation cannot be predicted accurately at this time because of the uncertainty of the severity of the events and the response of the

structures and banks to the events. Repair of structures to control upland drainage must not be overlooked.

6. Summary of Findings. Most of the structures constructed under the nationwide program have not experienced critical events to permit an assessment of their behavior and effectiveness. A few still are under construction at the time of this writing. Therefore, only generalized and preliminary evaluations can be formulated and presented.

a. Areas left unprotected between or above protection structures are frequently subject to erosion. This develops in a scalloping effect between hardpoints which can be held to acceptable limits by proper spacing of the hardpoints along the riverbank. Mulching, turfing, and other vegetation systems should be specified to provide a protective slope where possible. The vegetation should be planted at the beginning of the growing season and repaired as necessary until established. Wire mesh matting is ineffective in providing interim erosion protection for establishment of vegetative growth.

b. Floating ice and debris uplift and push down timber and wire fence dikes and Kellner Jack fields. They also submerged the floating tire breakwater. Stone used in the outer edge of a few rock hardpoints was dislodged by the ice jams and flows.

c. Piping of soil on drawdown or from normal perched water flows requires a filter material to prevent the washing of the natural soil through the blanket protection provided.

d. Upland drainage from interior areas must be controlled. The water must not be permitted to flow over and down the slope. Outlets should be provided where required to conduct the water to the river.

e. Rock toe protection when constructed with low cost upslope systems generally functioned satisfactorily against the events experienced to date.

f. Vandalism in urban areas is a problem and should be considered in the design and selection of protective systems. Fires, cutting with knives, etc., should be anticipated.

g. Geotechnical problems associated with a specific site must not be overlooked and an adequate design provided to maintain slope stability for the normal condition, the eroded channel bed condition, and drawdown conditions.

h. Trees used as pendants adjacent to the riverbank are difficult to transport to the site without damage to the roots and limbs needed for protection. The effectiveness of this type protective system remains to be evaluated.

7. On the following pages, reports for each of the projects are presented. They provide detailed information on placement, costs, and (to the extent possible) the effectiveness of the various methods of bank protection utilized.

YELLOWSTONE RIVER, INTAKE, MONTANA TO MOUTH

Section 32 Program Streambank Erosion Control Evaluation and Demonstration Act of 1974

YELLOWSTONE RIVER, INTAKE, MONTANA TO MOUTH DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

A. PROJECT NAME AND LOCATION

The Section 32 Authorization designated Streambank Erosion Control Demonstration Projects on the lower Yellowstone River with specific sites located between Intake, Montana, and the Mouth as shown on plate 1. The specific project sites are Cheney Creek Area and Horse Creek Area, North Dakota, and River Road Area, Montana. The Horse Creek Project was designed but not constructed due to funding limitations.

B. AUTHORITY

The project has been authorized under P.L. 93-251, the Water Resources Development Act of 1974, Section 32, "Streambank Erosion Control Evaluation and Demonstration Act of 1974," as amended by P.L. 94-587, the Water Resources Development Act of 1976, Sections 155 and 161; and the Missouri River Division letter dated 10 December 1976, subject, "Section 32, Streambank Erosion Protection Program."

C. PURPOSE AND SCOPE

Since the legislation specifically instructs the Corps of Engineers to implement multiple erosion control demonstration projects on the lower Yellowstone River, this report provides a general discussion of the characteristics of the river reach itself, describes bank erosion problems, and the types of bank protection selected for construction. The mandate does not automatically provide for a comprehensive erosion control program on this river

reach, but is a program to develop methods of preventing undesirable streambank erosion.

D. PROBLEM RESUME

Three priority project sites were selected from twenty-four identified erosion sites. The sites were selected based upon an evaluation of the following erosion related concerns, activity and shape of erosion, environmental factors, and benefits to public or private improvements. The sites are all located along the right bank of the Yellowstone River.

The River Road site is located on a side channel of the Yellowstone River. Severe erosion along this side channel bankline threatened a county road and a state irrigation pipeline with eventual damage. The county road and parallel irrigation pipeline are located at the base of high bluffs. Further erosion of the bankline would eventually require the relocation of these facilities.

The Cheney Creek site is located on a concave bend which had a highly active eroding bankline. Severe erosion at this site has required that a county road be moved approximately 250 feet landward to the base of high bluffs. If nothing was done to stop the erosion along the bend, the road would either have to be abandoned altogether or considerable expense would be incurred to relocate the road in the bluffs.

The Horse Creek site is located on a generally straight alignment suffering severe erosion which would be suitable for testing specific erosion control methods. The land along the site is irrigated, with two farmsteads next to the river bank.

II. HISTORICAL DESCRIPTION

A. GENERAL DESCRIPTION

1. GEOMORPHOLOGY.

a. **Physiography.** The Yellowstone Subbasin is located in the west-central portion of the Missouri River Basin and includes all the land drained by the Yellowstone River and its tributaries. The total drainage area of the Yellowstone River at its confluence with the Missouri River is approximately 70,000 square miles and includes portions of three states: Montana, Wyoming, and North Dakota. In terms of acres, this subbasin drains approximately 45.2 million acres of land, of which 50 percent lies in southern and western Montana, 49 percent in northern and central Wyoming, and 1 percent in west-central North Dakota. The drainage area at the upstream end of our study reach at Intake, Montana, is 66,800 square miles.

The Yellowstone River lies along the northern boundary of the drainage area so it does not have main left-bank or northern tributaries. Consequently, there are no large tributaries draining southward into the Yellowstone River. The principal southern tributaries include Clarks Fork and the Bighorn River, which flow northeastward from headwaters in northwest Wyoming; the Tongue River, which drains the central portion of the subbasin northeastward; and the Powder River, which flows northeastward through the eastern portion of the subbasin. Plate 2 shows the subbasin and its major tributary streams.

The Bighorn River Basin is the largest of all tributary basins, and drains more than the upstream portion of the Yellowstone River itself. The Bighorn drains almost 23,000 square miles, of which 19,626 square miles lie upstream from Yellowtail Dam and Bighorn Reservoir. With the exception of the Bighorn River, all major streams in the Yellowstone River Basin are relatively free-flowing streams.

b. **Topography.** The topography is essentially of four types. The most extensive is a region of plateaus, plains, basin, and isolated mountain ranges in the eastern portion with elevations ranging from 5,000 feet to 8,000 feet (mean sea level). Parts of the western and central portion of the subbasin are characterized by rugged mountains, broad valleys, and remnants of high plateaus at elevations of 4,000 feet to 14,000 feet. A small area in the extreme northeastern portion of the subbasin is a predominantly smooth plain with elevations ranging from 1,500 feet to 3,000 feet.

c. **Geology.** The lower reach of the Yellowstone River is within the Great Plains Physiographic Province. The valley is along the western margin of the Williston Basin, a geologic structural feature.

The Williston Basin has been part of the continental interior. Since pre-Cambrian time, it has functioned as a broad shelf area, marginal to the geosyncline on the west. The basin received predominantly marine-type sediments during the Paleozoic, Mesozoic, and Early Tertiary Eras. Sedimentation has been nearly continuous, except during minor periods of regional uplift and retreat of the seas. Some 17,000 feet of deposits have accumulated. These sediments are not grossly faulted or folded; bedding dips gently to the center of the basin indicating the shelf has remained undisturbed by major compression or orogenic activity.

Northeastern Montana became an area of erosion from the Early Tertiary to the Pleistocene Age. Prior to the Pleistocene continental glaciation, it is believed that the Missouri and Yellowstone Rivers flowed separately north through Canada to the Arctic Ocean, dissecting a broad valley into the bedrock. With the southward advance of glacial ice, both rivers were dammed. The Missouri River took an eastern route through Montana, intercepted the Yellowstone River, and then turned to the south...on its way to the Gulf of Mexico.

With the onset of Pleistocene continental glaciation, a Wisconsin lobe of ice advanced from the north down the Yellowstone River valley about 40 miles,

as measured from the Montana and North Dakota state line, and a lake was formed along the valley south of the lobe. Today, one finds a meandering river flowing on sand and gravel deposits, an indication of a geologically old valley. Another indication of this geologic age can be seen in the valley walls. The erosive action of the river has exposed marine sandstone and shale of the Fort Union Formation of Paleocene Age, in addition to gravel terraces and outwash deposits from the Pleistocene glaciation.

In the study reach, the Yellowstone River valley is nearly straight, bearing north-northeast flowing along the east, or right side (looking downstream) of the valley floor. From Intake, Montana, to the confluence with the Missouri River, the valley is 54.5 miles long. But the longitudinal slope of the valley changes abruptly, approximately 25 valley miles upstream from the Missouri River confluence. Upstream from this point the slope of the valley floor is 3 ft/mi, whereas downstream it is only 1 ft/mi.

In the upstream 12-mile reach near Intake, the average width of the valley floor is only 1.5 miles. It narrows to a minimum width of 1.0 mile and has a maximum width of slightly more than 2.0 miles. The average width of the valley floor in the central portion of the study reach is 2.0 miles. Here the valley floor ranges from 1.0 mile to 2.5 miles in width. In the downstream 12-mile reach adjoining the confluence, the valley floor is widest, averaging 3.5 miles. The maximum width is 4.5 miles and the minimum width is approximately 1.0 mile. Overall, the valley floor averages 2.2 miles in width.

2. VALLEY LAND USE

The subbasin encompasses approximately 44.9 million acres of land and 345,000 surface acres of water. Some 34 percent of the land is in Federal ownership. The U.S. Forest Service and Bureau of Land Management have the largest holdings, administering 5.8 and 7.6 million acres, respectively. A portion of the Yellowstone National Park occupies another 1.3 million acres within the subbasin. Approximately 29.8 million acres are in private, State,

or county ownership. Three Indian reservations (the Crow and Northern Cheyenne in Montana, and the Wind River in Wyoming) occupy nearly 3.9 million acres of Government trust land.

About 72 percent of the land is in pasture and range, 9 percent in forest and woodland, 7 percent in cropland, less than 1 percent in transportation routes and urbanization, with the remaining land in other uses. Of an estimated 3.7 million acres of irrigable land, approximately 1.3 million acres were irrigated in 1975. Irrigated acreage is expected to increase to nearly 1.5 million acres in 1985 and to about 1.7 million acres by the year 2000. These increases represent gains of 14 and 30 percent over current levels. All other agricultural land uses should remain near current acreages. Urban expansion and transportation land use are projected to increase nearly 20 percent by 1985, and more than double by the year 2000.

Of the 7 percent of land in crops, the major dryland agricultural products are wheat, small grains, and hay. Crops grown on irrigated lands include sugar beets, dry edible beans, hay and vegetables. These account for more than one-third of the cropland acreage. Timber production is important locally to several areas and production volumes have increased since 1962. More importantly, however, a large percentage of forest area also serves as grazing land for livestock. Livestock production is the most important source of agricultural sales receipts.

Recent increases in demand for northern Great Plains coal and lignite have transformed the subbasin and adjacent coal areas into an expanded energy development area. Current estimates place recoverable coal and lignite reserves at about 125 billion tons. Uranium, natural gas, and crude petroleum reserves, including oil shale and tar sand deposits, also exist. In addition, the subbasin contains about 80 percent of all chromite deposits known to exist in the western hemisphere.

Continued growth in significant industrial areas is anticipated. These include the manufacturing of energy-related products and food processing as

well as the tourist industry. Energy products development is predicted to dominate all other manufacturing activities. However, the availability of and access to scenic attractions and recreation facilities is expected to continue to contribute to the economy and encourage increased urbanization in certain locations.

3. HYDROLOGIC CHARACTERISTICS

Because of the diverse topography, there is a wide variety in climate. In general, the average annual precipitation range is from 8 inches to 40 inches in the mountains and 13 inches on the plains. Large winter accumulations of snowfall are common in the mountains. The average annual lake evaporation ranges from 28 inches to 42 inches in the mountains and is about 38 inches on the plains.

Temperature anomalies are normal in the mountains, but January is usually the coldest month and July the warmest. Mean daily maximum temperatures (in degrees Fahrenheit) vary from 20 to 36 degrees in the mountains and 28 degrees on the plains in January, and 72 to 92 degrees in the mountains and 90 degrees on the plains in July. Mean daily minimum temperatures vary from 0 to 12 degrees in the mountains and 4 degrees on the plains in January, and 40 to 56 degrees in the mountains and 57 degrees on the plains in July.

The frost-free period varies from 30 to 90 days in the mountains, depending on the elevation, and averages 130 days on the plains. In the mountains an average of between 2,400 and 3,000 hours of sunshine occur annually, while the plains receive about 2,900 hours.

4. EXISTING CHANNEL CONDITIONS

The Yellowstone River is noted for its scenic beauty. Beginning from the slopes of Younts Peak in Wyoming, the river enters Yellowstone National Park and feeds into Yellowstone Lake. Below the lake, it plunges 422 feet in two

spectacular waterfalls and enters the Grand Canyon of the Yellowstone. Leaving the park at Gardiner, Montana, the river turns north to Livingston and follows a northeasterly meandering and island braided course to its confluence with the Missouri River at Ft. Buford, North Dakota. For the existing channel conditions of the demonstration reach see subsection B, DEMONSTRATION PROJECT REACH, on page 12 of this report.

5. ENVIRONMENTAL CONDITIONS

Several aspects of the environment of the lower Yellowstone River are briefly discussed in this section. These topics include vegetation, fish and wildlife, water quality, and the impact of man. Discussion of vegetation and its role in the environment of the lower Yellowstone River is limited to the following specifics: the importance to wildlife of the successional stages of habitat in the immediate vicinity of the river; and the influence of the river on vegetative succession as it applies to that habitat.

a. Vegetation. Vegetative succession in the river environment is not only highly dynamic due to changes in the river, but also shows no uniformity in rate or direction. This is readily discerned from observing the various stages of growth found near the river. Mature cottonwood gallery forests (cottonwood/green ash/poison ivy) occupy locations that are high above and/or distant to the river; these are exposed to infrequent flooding. Secondary cottonwood gallery forests (cottonwood/rose/brome grass) are found in positions intermediate to those of mature forests and the pioneer stands. These are possibly the most dynamic forests since they accumulate biomass at a very rapid rate. Rapid growth of vegetation in these forests is the result of replenishment of the nutrient pool during inundation where water velocities are not excessive. Pioneer cottonwood gallery forests (cottonwood/brome grass) occur on most near-bank locations where sedimentation has built a substrate suitably stable for the establishment of the cottonwood seedlings.

Pioneer willow/sedge stands are found on recently deposited alluvial areas. Mesic grassland (Canadian wild rye/brome grass) stands occur in swail areas near the river channel and in swails and old chutes that are inundated

during high water. Dry grassland stands (western wheat grass/reedgrass) occur on old river chutes that have been filled and elevated by sedimentation. Secondary dense mesic shrublands (rose/dogwood/green ash) occupy old river chutes with high ground water or periodic saturation during flooding.

Secondary Russian olive woodlands (Russian olive/snowberry) are infrequently encountered along sections of the river where the channel is straight and/or wide. Mature willow forests (peach-leaved willow/shining willow) form thicket-type stands within the mature and secondary cottonwood gallery forests in areas where old river chutes predominate. Climax green ash forests (green ash/poison ivy) occur as small isolated woodlands on large islands. Climax American elm forests (American elm/green ash/American plum/Canadian wild rye) occupy high bank positions along the downstream reaches of the river.

b. <u>Mammals</u>. The riverine environment provides important habitat for a variety of terrestrial mammals including white-tailed deer, beaver, weasels, skunk, muskrat, and porcupine. Beaver use the young willow and cottonwood in the near-bank areas. They also use more mature cottonwood, as well as elm, ash, and dogwood. Skunk forage for insects and amphibians along the river and eat the fleshy fruits of shrubs, such as chokecherry, American plum, and grape. Porcupine use a wide variety of shrubs and herbaceous vegetation throughout the ecosystem. During the winter, porcupine subsist largely on the bark of ash and cottonwood.

White-tailed deer use all of the habitat types present in the riverine system. The dense shrub understory of some mature cottonwood gallery forests and secondary dense mesic shrublands provide cover and bedding ground. The open parkland cottonwood stands provide a great deal of desirable edge habitat as well as forage during the summer months. Species common to the riverine habitats that are included in the diet of the white-tailed deer are willow, serviceberry, snowberry, dogwood, cottonwood, green ash, rose, wheatgrasses, and reedgrasses.

c. **Avifauna.** Avifauna are also important users of the lower Yellowstone River. Migratory waterfowl species include Canada geese, great blue heron, mallards, mergansers, and white pelican. Numerous passerines common to the area include swallow, starling, robin, hairy woodpecker, flicker, and veery. Upland game birds found in the area include pheasant, sharp-tailed grouse, Hungarian partridge, and mourning dove. Songbirds in the riverine habitat are largely insectivorous, although some seed-eaters are found. Both groups are most common in the habitats with well-developed shrub vegetation.

Migratory birds use the riverine habitat for nesting, fishing, loafing, and feeding. Substantial populations of Canada geese use the river throughout the year, although wintering populations are low. Geese nest on the larger islands and use the sparsely vegetated sandbars as loafing and feeding areas. Large migratory populations of dabbling ducks use the riverine habitat in a similar fashion. The agricultural areas are also an important feeding area for all migratory waterfowl.

d. **Aquatic Life.** The Yellowstone River supports a trout fishery in the upper reach and a warm-water fishery in the lower reach. Diversity of species increases progressively downstream. Eleven fish species have been recorded in the upper river, 20 species have been collected in the middle river, and 46 species have been collected in the lower river. The major fish species found in the lower Yellowstone River include channel catfish, burbot, shovel nose sturgeon, goldeye, and sauger. Other fish found in the river include northern pike, paddlefish, silvery minnow, stonecat, small mouth bass, walleye, freshwater drum, white crappie, and white sucker. The food source for this fishery is predominantly benthic invertebrates. Mayfly, caddisfly, and other fly larvae dominate the bottom fauna.

The aquatic environment, like the terrestrial portions of the riverine system, is directly influenced by the river. The quality of specific habitats are altered by changes in river channels and flow cycles. These habitat cycles are natural phenomena, as are the point source degradations of water quality due to bank erosion. Population numbers and species diversity of

fish, benthos, periphyton, planktonic algae, and rooted aquatic vascular plants are in phase with the random cycles or patterns of the river and are dependent on these patterns.

e. **Endangered/Threatened Species.** In accordance with Section 7(c) of the Endangered Species Act, as amended, a list of endangered and/or threatened species occurring in the project area was requested from the U.S. Fish and Wildlife Service (USFWS). The USFWS has determined that the endangered bald eagle and whooping crane may occur in the project area.

- **Whooping Crane.** The Whooping Crane Tracking Project Coordinator from the USFWS Pierre Area Office was contacted for a list of whooping crane observations in Montana. The closest reported sightings to the lower Yellowstone River were 25 miles southeast of Medicine Lake National Wildlife Refuge (NWR) (1963 sightings). Medicine Lake NWR is approximately 50 miles northwest of the Horse Creek Area. The most recent recorded sighting in northeastern Montana was 4 miles southeast of Medicine Lake NWR.

- **Bald Eagle.** Biologists from the Montana Department of Fish and Game were consulted for their knowledge of the presence and distribution of bald eagles on the lower Yellowstone River. The State biologists stated that migrating bald eagles are common along the river, especially in December and March. No eagle nests are known to exist from Intake to the mouth of the river. Some bald eagles winter-over along the river depending on ice-free conditions and winter mildness. Distribution of the eagles is wide because of the many available cottonwoods.

f. **Water Quality.** The water quality of the Yellowstone River is generally good. There is some degradation of quality downstream from the cities; this is adversely impacted in lower reaches by the introduction of sediment, turbidity, and warm water from irrigation return flows. During the spring runoff period the river is generally quite turbid and carries a heavy sediment load. Much of this sediment results naturally because of unstable soils within the drainage basin. Prevailing land use practices throughout the

basin, however, also appear to contribute significant quantities of sediment, both from overland erosion and increased bank erosion on tributaries and the main stem.

g. Impact of Man. All parts of the present riverine system have been influenced by man's activities. Principal among these are the effects of agriculture in altering the higher topographic portions of the flood plain. Secondary are the effects of grazing on the remaining gallery and climax forests. With the exception of near-bank willow stands, these forests show the effects of grazing by sheep and cattle.

B. DEMONSTRATION PROJECT REACH

1. HYDRAULIC CHARACTERISTICS

a. Existing Channel Conditions. The Yellowstone River from Intake, Montana, to its confluence with the Missouri River is a meandering river flowing in a broad valley on sand and gravel deposits and braiding its way through many islands. Within the seventy-mile reach there are two highway bridges and one railroad bridge. Portable and permanent irrigation facilities are scattered throughout. The diversion dam at Intake, Montana, is the only dam on the river. It supplies water through a system of canals and laterals along the left bank for the federally funded Lower Yellowstone Irrigation Project. In the reach from Intake, Montana, to the bridge on Montana State Highway No. 23, near Sidney, the lower Yellowstone River is an island-braided river. Islands and vegetated bars divide the river into as many as six channels at one cross section. At another, there are three channels, all of equal width. This segment is designated the "steep" reach of the lower Yellowstone River because the river slopes approximately 2 feet per mile.

Downstream from the Montana State Highway No. 23 bridge the lower Yellowstone River changes form. Although there are numerous sandbars and point bars, the main channel is usually large and the side channels are very small.

Islands are not so numerous. This reach is designated the "flat" reach of the lower Yellowstone River. The river slope is approximately 1 foot per mile.

Composites of historical river location maps for the river reach are shown on plates 3 through 8. These maps show the 1978, 1951, and 1938 river locations, based upon aerial photographs, overlayed on 1965 base maps drawn from U. S. Geological Survey quadrangle maps.

b. **Stage Fluctuations**. The hydrologic data analyzed in the test reach is from the gaging station on the Yellowstone River near Sidney, Montana. This gaging station is run by the Water Resources Division of the U. S. Geological Survey.

For the period of monthly streamflow records from 1911 to 1977 for the gaging station, the mean annual discharge is 9.42 million acre-feet per year. Annual discharge ranges from low values or 4.21 million acre-feet in 1934 and 4.26 million acre-feet in 1961, to a high value of 15.43 million acre-feet in 1924.

Plate 9 indicates that of the eleven annual peak discharges in excess of 100,000 ft^3/s, six occurred during the first one-third of the period record, three during the middle third of the record, and only two during the last third. Another comparison between the early, middle and last third of the record was made by averaging the five largest floods during each period. The five largest floods, from 1911 to 1933, averaged 127,000 ft^3/s. During the middle third of the record, 1934 to 1956, the five largest floods averaged 114,000 ft^3/s. From 1957 to 1978, the average of the five largest floods was only 91,300 ft^3/s.

A downward trend in the very large annual floods is apparent from these flood records. This trend must be related to climatic trends, changes in land use, new storage reservoirs, or a combination of these factors. In controlling about 28 percent of the watershed, Yellowtail Dam should have had a

significant effect on the annual peak floods at the Sidney gage since 1965. From 1965 to 1978, the average of the five largest floods was 89,400 ft^3/s.

Further analysis of the data available from the Sidney, Montana, gaging station reveals the following:

- Four of the five maximum discharge years occurred prior to 1928.

- The annual cycle of streamflow with peak monthly values almost always occurs in June. This is the month of peak snowmelt runoff. August through February is the normal low-flow period of the year.

The only identifiable change in the pattern of monthly runoff occurred starting with the 1968 water-year. Since that time, the low-flow discharge at the Sidney gage has been significantly higher than at any prior time. The reason for this abrupt change is apparently due to low-flow augmentation from Bighorn Reservoir behind Yellowtail Dam. The Bighorn River Basin contributes approximately 30 percent of the annual streamflow in the Yellowstone River. The increase in low-flow discharge since 1968 is also readily seen in the daily flow-duration curves on plate 10.

Concerning the relationship between annual peak discharge and annual river level or river stage, it should be mentioned that peak discharge and peak river stage do not always occur at the same time. Occasionally, the peak stage of the river during the year occurs when ice jams cause backwater. For example, the records show the gage height of the peak annual discharge in the 1971 water-year was 14.97 feet on June 28, 1971, but the annual maximum height of 21.67 feet occured on February 17, 1971, due to backwater from ice jamming downstream.

The ice and the ice jam problem that usually occur in March are especially severe in the lower reaches of the Yellowstone River. Ice up to 40 inches thick and ice jams 10 to 30 feet high form in the river. Because the river flows from southwest to northeast, the ice breaks up in the upstream

reaches earlier than in the lower reaches. Spring thaws to the south cause the rise in river stage to reach the lower Yellowstone before the ice cover is melted. The rising river then causes the ice to break up and jam on the many islands and side channels. Ice jams result in new channels being developed, high river stages, and damage to structures along the river.

e. **Sediment Characteristics.** Annual estimates of suspended sediment load in the Yellowstone River at Sidney, Montana, have been made since 1939. Sediment data through the 1976 water-year (ending September 1976) were analyzed. Annual values of suspended sediment load are plotted on plate 11. The mean annual suspended sediment concentration, in milligrams per liter, is plotted on plate 12.

Annual sediment load at the Sidney, Montana, gage has ranged from a high of 62 million tons in 1944, to a low of 5 million tons in 1966. This is equivalent to a sediment yield ranging from 72 to 900 tons per square mile per year. Mean annual sediment concentration was a maximum of 5,421 milligrams per liter in 1941, and a minimum of 652 milligrams per liter in 1966.

A prominent downward trend in both annual sediment load and sediment concentration is evident from the plotted data. In addition to this trend in the historic data, a significant difference in annual sediment load is apparent starting with the 1949 water-year. This change is readily seen on plate 13. On this plate, a straight line average through the 1949 to 1976 portion of the curve is much flatter than the 1939 to 1948 portion, indicating a decrease in sediment load for the 1949 to 1976 period.

2. RIVERBANK DESCRIPTION

a. **General.** The riverbank material is generally silt, with occasional areas and lenses of sand or clay. The river occasionally flows next to high bluffs composed of varying sandstone layers with occasional coal layers interspersed. The bank slopes range from 45° to nearly vertical.

Construction materials suitable for the Yellowstone River erosion control structures are stone, sand and gravel. Existing sources of field stone are mostly located in northwestern North Dakota, within fifty miles of Sidney, Montana. There are no quality stone quarries within reasonable distance from the test reach. Sand and gravel quarries are scattered relatively close throughout the reach.

The vegetation occurring along the test reach consists of cottonwood forests at near bank locations and at various stages of development; young willow-sedge stands adjacent to sand and gravel bars; grasslands near the river channel and in swales and old chutes consisting of reedgrass, Canadian wild rye, brome and some wild licorice; American elm forests occupying high bank positions; green ash forests as small isolated woodlands on large islands; Russian olive woodlands at high bank locations; and mature willow forests throughout the study reach.

Existing streambank erosion control structures installed by local land-owners and agencies consist of anchored car bodies, concrete rubble, and some stone riprapping. Since 1965, the Bureau of Water and Power Resources has also constructed bank stabilization works at local isolated areas along the left bank. The Bureau used rock riprapped hardpoints, steel jacks and gravel blankets on sloped banks to stop erosion where it was threatening lateral canals in the Lower Yellowstone Irrigation Project.

Although the Yellowstone River has some bankline erosion protection, it is still basically uncontrolled. Its river banks move intermittently most of the time. In one year, a large movement may occur; then in the following several years no erosion may occur at all. Furthermore the record for some banklines shows periods of deposition as well as erosion.

b. **Project Sites.** The following describes the riverbank and erosion situations at the River Road and Cheney Creek project sites.

The site plan of the River Road project area is shown on plate 14. Photos 1 through 3 show the project area and erosion bankline. At the side channel offtake, the bed is gravel and the velocity is approximately 6 feet per second, even at low flow. The side channel accounts for approximately 5 percent of the river flow. At the upstream end, the top of the bank is 8.5 feet above a water surface elevation representing 10,000 cubic feet per second. This area is considered a high flood plain and is covered with a new layer of sediment. According to geomorphic studies, the river was into the present flood plain area in 1938. The area also has a considerable number of trees which are less than 50 years old. The low flood plain is approximately 2 feet lower than the high flood plain. Up to 6 inches of new sediment have been deposited on the low flood plain. Unlike the high flood plain, there are no trees in the low flood plain area.

Further downstream along the side channel a road and underground irrigation pipeline follow along the bank at the base of 100-foot high bluffs. The road and pipeline are threatened by severe erosion at the bend. At the road, the bank height is 25 feet above water surface.

Downstream of the side channel the bank has a small flood plain with new sediment deposits. This flood plain is approximately 10 feet above the water surface. The bank material at the River Road site consists of fine and coarse sand and silt.

Historic banklines at the River Road site are shown on plate 15. In 1883 the right bankline in this area was nearly straight. In 1930, a smooth, long radius bend was apparent. This bend migrated to the right toward the bluffs and developed a sharp and irregular shape by 1938. Large changes upstream resulted in a great movement away from the bluff between 1938 and 1951. In one spot, the 1951 bankline was approximately 1/2 mile farther north than in 1938. Since 1951, the maps indicate a slow, systematic erosion back towards the road, so that in 1978, the riverbank was at the toe of the road.

The unusual, nearly perpendicular angle the side channel has formed with the main channel flows is an anomaly. Erosion in this area is mainly caused by the sharp angle of attack during high and intermediate flows which occur from March through July. However, ice and ice jams also add erosion stresses to the bankline during the winter months and the period of spring thaw.

Plate 16 shows the Cheney Creek area site plan. Photos 10 through 14 show the erosive bankline. The bankline at the structure location is a relatively low terrace approximately 19.5 feet above the water surface elevation representing 10,000 cubic feet per second. The terrace is cultivated but not irrigated. There is a 4-foot upper silt and clay layer that covers a 2-foot coarse sand layer on the upper bank. A deep fine sand layer slopes toward the river. The downstream bankline is an older terrace 25.5 feet high with vertical banks. There is a 6-inch to 18-inch clay layer covering the sand. The terrace is cultivated and irrigated.

Studies indicate that the Cheney Creek area is historically one of the highest active erosion sites. Erosion rates determined by comparing the available surveys and aerial photography of the river, show that the greatest erosion occurred along the bankline where the old Cheney Creek Road once paralleled the river and has since eroded away. See plate 17. At the project site segment, from 1901 to 1978, the river has eroded 1,800 feet landward, averaging 23 feet per year. Upstream from this site, a 3,000-foot segment ending at the Montana-North Dakota state line also has experienced severe erosion, but to a lesser extent. Between 1974 and 1978, erosion at the project site averaged 50 feet per year and by 1978 erosion had progressed 200 feet landward.

Bank erosion at the Cheney Creek site is caused by the interplay of several factors, among which include soil characteristics, angle and velocity of the current. The soil at the site lacks coehsion. The bankline is composed of various soil layers and lenses generally consisting of a thick coarse sand layer capped by a fine sand layer with a thin layer of clay on the surface. The angle and velocity of the current also trigger erosion.

The concave shaped bankline and river velocities of 6 to 8 feet per second are conducive to the removal of materials at the water surface causing the upper bank to slump or slip. Many times wind and wave action work in combination with the velocity. In addition, ice gouging and ice jams damage the bank during the winter thaw. Large chunks of ice are flipped onto the bank terrace due to the tremendous pressures. The ice jamming is attributable to the very short, sharp narrow bend 3 miles downstream. Normal runoff from the high bluffs and flood irrigation practices also keep the bank materials in a saturated condition.

III. DESIGN AND CONSTRUCTION

A. GENERAL

The Streambank Erosion Control Evaluation and Demonstration Act of 1974 authorized and directed the Corps of Engineers "to develop methods and techniques to prevent and control streambank erosion." Maintaining or enhancing the wildlife and aquatic habitat was also a principal concern. This legislation was not designed as an operational authority for correction of streambank erosion problems.

Preliminary investigations and evaluations at potential sites were conducted. As a result of these investigations and evaluations contract design plans and specifications were prepared for three demonstration sites along the lower Yellowstone River. These sites were specifically designated as high priority areas. The sites were the River Road Area in Montana (river mile 28.9) and the Cheney Creek Area (river mile 19.8 to 20.4) and the Horse Creek Area (river mile 11.2 - 12.3) both in North Dakota. The River Road and Cheney Creek projects were completed in December 1980. The Horse Creek project was not constructed due to limited funding.

B. BASIS OF DESIGN FOR EACH TYPE OF PROTECTION

1. ENGINEERING OBJECTIVES

Engineering objectives are those goals established to provide perspective and scope to individual project formulation and design. The engineering considerations necessary to achieve these objectives are listed below.

- Least-cost, multipurpose problem solutions
- Materials
- Construction techniques
- Structure type, location, and orientation
- Minimize potential future maintenance costs

2. ENVIRONMENTAL OBJECTIVES

These are environmental considerations taken into account in the formulation and general design of individual projects.

- Minimize woodland clearing or the disturbance of any other sensitive or unique habitat;
- Protect important or critical habitat;
- Avoid disturbance of endangered fish and wildlife species during construction;
- Create desirable aquatic habitat with structure configuration and various types of structure materials;

• Consider structure designs that improve pedestrian and wildlife access to the water's edge;

• Preserve the natural appearance and esthetics of the waterway; conceal structures with topsoil and natural vegetation; low profile structures are generally less noticable;

• Avoid destruction of or protect cultural resources, as appropriate.

3. SITE SPECIFIC OBJECTIVES

a. **River Road.** The River Road site represents a streambank erosion problem area ideally suited to utilization of the sandfill dike. See plates 18 and 19. A stone revetment was considered as a solution but the sandfill dike was selected for the following reasons:

• Construction of a stone revetment would require working space along the road between a steep bluff face and the adjacent, very high riverbank. This situation would restrict the contractor's operations, requiring special traffic regulations, extra safety precautions and associated higher costs.

• The sandfill dike is more economical. Constructing a stone revetment along the eroding bank is economically comparable to the sandfill dike only if the revetment is built in segments, with substantial unprotected gaps left between the segments. This alternative has adverse impacts on the natural bankline and adjacent high-bank area. These impacts are caused by the clearing, excavation and other construction activity required in the revetted areas, and by continued erosion in the gaps. A stone revetment provides little or no improvement in aquatic habitat. The river would probably continue an established pattern of alternately concentrating along the bank, causing severe erosion and then shoaling in the area.

• The sandfill dike can be designed to any desired elevation, thus limiting or preventing flood waters from entering the backwater area created by the dike.

• The water surface area affected by the proposed structure is minuscule compared to the water surface of the adjacent main channel area.

• The new backwater area will provide additional highly desirable aquatic habitat.

• The sandfill dike can be constructed to a low profile, with all visible surfaces at relatively flat slopes and with vegetation and/or gravel covering. If properly constructed at a relatively low elevation, the low profile should allow ice to pass over the structure, thus reducing damage and maintenance costs.

• A structure similar to sandfill dikes was one of the 6 primary erosion control techniques originally recommended in the Missouri River Demonstration Program. The proposal to construct a sandfill dike as a demonstration is not in conflict with any goals or criteria for either the erosion control, the environmental or recreational objectives. Evaluation of all erosion control techniques considered in several "Streambank Erosion Control Demonstration" reports indicates that the sandfill dike meets more of the stated and implied criteria and goals than any other single technique.

b. **Cheney Creek.** An actively eroding concave bankline existed at the Cheney Creek site. See plate 20. Two structural alternatives were considered which could solve the erosion situation; a system of stone hardpoints or a stone revetment.

The alignment at Cheney Creek and the active erosion occurring there indicated a stone revetment structure was the most practical and best suited solution. A system of hardpoints was rejected as a solution because of economics. The radius of the concave bankline required close spacing of the hardpoints which increased the material quantities far above those quantities required for a conventional stone revetment.

A special stone revetment was designed for testing at the Cheney Creek site. Due to the locally high stone costs, a structure was designed that

would minimize stone use. The stone revetment placed along the bankline was designed at a low elevation which was considered minimum protection for most of the river flows that occur. The upper bank was sloped back and reinforced with stone segments placed on the sloped bank from the revetment perpendicular to the flow. Extensive vegetative treatments were incorporated throughout in a manner to also structurally reinforce the bank yet create a visually attractive project. This structure was named reinforced composite revetment.

C. CONSTRUCTION DETAILS AT EACH DEMONSTRATION PROJECT

1. RIVER ROAD AREA

The construction consisted of approximately 600 linear feet of sandfill dike placed across a side channel as indicated on plate 19.

The dike was built up to the level of the flood plain on the right bank across a side channel to a vegetated bar. The sandfill dike had a crown width of 10 feet with 5 horizontal to 1 vertical side slopes. It consisted of sandfill with a central gravel core. The surface of the sandfill dike, exposed at a low flow was seeded, fertilized, mulched, and covered with a 3-inch gravel blanket. Sprigs of bullrushes were then planted through the gravel blanket. Photos 4 through 9 show the construction of the sandfill dike through the completion of the structure.

Approximately 700 cubic yards of clean gravel from a local quarry were used for the inital closure of the side channel. The closure acted as a corridor for construction equipment. An estimated 6,140 cubic yards of sand were used to construct the sandfill dike. The sand fill came from a dry borrow area on the north side, nonvegetated portion of the island which was connected by the dike. The borrow area was uniformly excavated to an approximate 1-foot depth for the necessary material quantities. The limited excavation depth prevents the borrow area from becoming a possible channel and permits the area to refill with sediment during the next high flows.

Approximately 330 cubic yards of gravel were used for the gravel blanket. The gravel came from the island borrow area previously described.

The vegetative treatment was arranged in alternating strips 20 feet wide and 50 feet long. The 20-foot dimension was measured along the axis of the dike and the 50-foot length extended 20 feet on each side of the 10-foot wide crest. The first (southwest) 20-foot strip was only seeded. The second 20-foot strip was seeded and sprigged. The system alternated along the length of the dike so all odd numbered strips had seed only and even numbered strips had both seed and sprigging. The sprigging was locally collected bull-rushes harvested by scything. Sprigging was planted through the gravel blanket into the upper sand portion of the dike. The seed, consisting of streambank wheatgrass, canary reedgrass, and prairie reedgrass was planted prior to placement of the gravel blanket.

2. **CHENEY CREEK AREA.**

The plans and sections for the Cheney Creek Area are shown on plates 20 through 24. The project consisted of constructing 2,400 linear feet of reinforced composite revetment. Reinforced composite revetment consisted of rock riprap toe fill placed along the eroding bankline with riprap or vegetative tiebacks placed strategically along the upper bank. A special upstream tie-back to prevent structure flanking and drains for surface runoff were also included.

Rock riprap was placed in the river along the toe of the eroding bank-line. See photos 15 and 16. The crown was capped with a 2-foot thick mixture of rock, random fill, and willows locally collected which were cut to approximately 24-inch lengths. Seed was then broadcast over the exposed riprap surface.

During the course of initial construction the topsoil was scraped and stockpiled. After installation of the toe fill the remainder of the bank was shaped to the desired slope. In shaping the slope, the following criteria were followed.

• The slope was irregularly shaped to follow the existing bankline. This maintained the natural shape characteristic of the bankline.

• Natural vegetation occurring along portions of the upper bank during slope construction were protected and transplanted into the tieback regions and into the new upper bank slope. Any vegetation not suitable for transplanting was placed in the earthfill at the toe of the bank.

• The bankline length was split nearly in half based upon differing slopes of 2.5H on 1V and 2H on 1V identified as Subreach B and Subreach C, respectively. Different slopes were selected to determine soil stability and vegetative development.

Tieback trenches were excavated every 30 feet along the bankline. The trenches were approximately 3 feet wide and 20 feet long and positioned perpendicular to the river flow. The trenches were alternately filled with riprap or vegetation. See photos 21 and 22. The riprap tiebacks were a 1-1/2 foot depth mixture of stone, random fill, and willow slash. The vegetative tiebacks were concentrated plantings of willows in 6-inch trenches of two forms; facines and wattles, or lengths of individual willows. Facines consisted of 1/2- to 2-inch diameter, 3- to 10-foot long willows packed together in tight continuous wired rolls 4 to 6 inches in diameter. The facines were staked into position along the trench with wattles. Wattles were 2- to 3-foot willow cuttings. The facines and wattles were then covered with random fill. The vegetative tiebacks also consisted of individual willows 2 to 8 feet in length layed flat in the trenches covered with random fill to the finished grade. In the spaces between the tiebacks, various vegetation types and transplant stock were planted. The upper bank areas were then seeded.

Two drains were installed on the structure to direct surface runoff. Each drain consisted of a gravel filter layer and riprap overlayed with gravel. The upper surface along the vicinity of the bankline was graded for proper drainage to these drains.

At the upstream end of the structure, a special stone tieback was constructed landward into the bank 70 feet and oriented 30° upstream from a line perpendicular to the river flow, to protect the structure from flanking.

Photos 17 through 20 show the completed project. Photo 23 shows an aerial view of the project area and the completed structure.

D. COSTS

Project Costs (Adjusted to 1981)

	Contract Structural $	E&D $	S&A $	Total $
River Road	51,960	63,456 2/	7,546	122,962
Cheney Creek	403,085 1/	73,707 2/	11,775 1/	488,567

Costs Per Lineal Foot of Structure

	Structure Length LF	Project Cost $	Cost/ LF Structure $
River Road	600	122,962	205
Cheney Creek	2,400	488,567	204

Costs Per Lineal Foot of Bankline Protected

	Bankline Protected LF	Project Cost $	Cost/ LF Protected $
River Road	3,500	122,962	35
Cheney Creek	2,400	488,567	204

1/ Cheney Creek contract price and S&A costs are not finalized pending the outcome of contract modifications. The contract price indicated includes the approximate modification costs.

2/ Engineering and Design includes A-E design costs.

IV. PERFORMANCE OF PROTECTION

A. MONITORING PROGRAM

1. **MONITORING SCHEDULE.** Monitoring schedules for the River Road Area and Cheney Creek Area Projects are shown on plates 25 through 28.

2. **MONITORING OF PHYSICAL FEATURES.** The bankline survey and hydrographic survey conducted during design established a baseline for monitoring physical features. Overbank and streambank cross sections were made during the construction periods. The cross sections were surveyed perpendicular to the structure alignment or perpendicular to the riverbank, depending upon the type of erosion control structure used. The data will document any structural movement and changes. Velocity measurements are scheduled annually along the Cheney Creek structure bankline. Aerial, boat, and onbank observations of the general river conditions, erosion activity, vegetation performance, and structural changes and performance at each demonstration site will be evaluated and noted, with emphasis on changes from previous observations.

3. **MATERIAL TESTS.** Qualitative and quantitative soil evaluations performed prior to construction at each demonstration site determined potential erodibility and stability of the soil. Observations are scheduled for each structure to judge the weathering durability properties of the various construction materials relative to extreme temperature changes, stage fluctuations, wave wash, and ice. The construction materials were sampled during construction to verify that the materials met specifications. The structure materials will be sampled periodically, as shown on the schedules, to attempt a quantification of changes in properties or gradation.

4. **PHOTOGRAPHY.** Oblique aerial photographs of the project area will continue to be taken and compared with existing oblique photos to verify changes from previous observations. Vertical controlled aerial photographs of the study reach were flown in November 1980. They will serve as a construction period baseline. Every two years, vertical controlled aerial photographs of the entire reach are scheduled. Such coverage will permit quantification of erosion losses, land use changes and other changes caused by the project or by natural events. Additional photographs of the project sites taken from established riverbank locations will verify any changes that may have occurred to the structures or bankline.

5. **BIOLOGICAL MONITORING.** The Corps of Engineers will conduct continuous monitoring of the demonstration sites, and concentrate on those conditions which significantly alter the aquatic or riparian habitat either negatively or positively. Reports will be prepared for each site to evaluate the habitat and impact of construction on fish and wildlife resources.

6. **REVIEWS.** The Corps of Engineers will perform an annual field inspection. Any concerned agency or entity wishing to review the conditions and effects of the demonstration projects may participate in the field inspection. Should the rate of changes appear rapid, intermittent inspections will also be conducted. Participants will be requested to formulate and submit their observations and conclusions to the Corps. A field report consolidating the participants' observations and conclusions will then be prepared.

B. EVALUATION OF PROTECTION PERFORMANCE

1. RECENT RIVER CONDITIONS

The River Road Area and Cheney Creek Area projects were both completed in December 1980. Each project survived, historically, one of the mildest

winters ever recorded. Little snow fell, a uniform icemelt and below average precipitation all accounted for a lack of the usual March ice jams. Also in April some of the lowest discharge readings at the Sidney, Montana gaging station were recorded with a low reading of 2980 cubic feet per second recorded on 30 April 1981. Flows began to increase in May, with above average discharges occurring in June with a maximum reading of 59,100 cubic feet per second on 13 June 1981. An evaluation of the effects of this first winter and spring for the project follows.

2. RIVER ROAD AREA

No damage due to ice or ice flows occurred to the sandfill dike. The sprigs of bullrushes planted in late November 1980 were showing initial signs of sprouting in May. The seeding has not sprouted yet. The channels, bankline and the island connected to the sandfill dike in the project area had not changed since the construction period.

The crown elevation of the sandfill dike is designed for a discharge of 70,000 cubic feet per second. The structure was not overtopped during the June rise. The dike functioned as designed, and successfully diverted flows from the side channel bankline. Some damage may have occurred to the seeding and bullrushes on the riverward slope of the dike, but only future inspections will verify this.

3. CHENEY CREEK AREA

Soon after construction of the stone revetment and sloped upperbank backfill, a slump occurred along the central portion of Subreach C (the downstream half of the structure) for approximately 500 feet. See Photos 17 and 18. The fill material slid approximately 6 to 10 feet vertically at the landward edge, and slumped riverward over the stone revetment. The revetment alignment remained stationary. Precise cause of the failure is not known, but is probably caused by the 1 vertical to 2 horizontal slope with uncompacted fill as specified for Subreach C. It was determined that the area

would not be repaired. The area was to be monitored for further slumping and for slumping of nearby areas. To date, adjacent upper bankline slump failures have not occurred.

The reinforced composite revetment and vegetative treatments at the Cheney Creek site were not damaged through the winter and early spring thaw. The vegetative tiebacks and upper bank vegetative treatments, consisting of various forms of willows and scattered locations of seeding showed the green evidence of life during May. The willow slash mixed with the stone crown of the revetment structure was not active.

The high flows in June approached elevations along the upperbank near the tops of the stone tiebacks and vegetative tiebacks. Their upper extension landward of the revetment was constructed at an elevation representing 70,000 cubic feet per second. Considerable damage occurred to the vegetative tiebacks and vegetative treatments between the tiebacks by the high flows since there was insufficient time for their establishment within the upperbank. Although there was some scattering of the stone in the stone tiebacks, their integrity remained intact. Erosion of the lower bank between the stone tiebacks created a curved bench. The stone revetment alignment had not changed since it was constructed.

C. ENVIRONMENTAL EVALUATION

The U.S. Fish and Wildlife Service evaluated the environmental aspects of the Yellowstone River demonstration projects and completed their report prior to 1 February 1981 as requested. At the time of the USFWS report, the structures were completed less than 2 months, so their evaluation could not incorporate a first-year experience. Their letter report follows as exhibit 1.

D. REHABILITATION

Rehabilitation of either project is not required.

E. CONCLUSION

The sandfill dike and the reinforced composite revetment survived a mild winter and low March melt flows that allowed some establishment of the vegetative treatments. Both structures functioned as designed through the relatively high June flows. The River Road and Cheney Creek projects have not been completed for a sufficient time to adequately evaluate their effectiveness. The true test will come when future ice jamming or flooding occurs at the Cheney Creek and River Road sites. Future monitoring of these projects should be maintained to determine the effectiveness of the designs, as local interest is quite high. These structures will surely serve as first class models for many future erosion control projects constructed by local landowners or local agencies.

PHOTO 1. Downstream Aerial View of Erosion Site Prior to Construction. (Photo taken 30 October 1980)

RIVER ROAD AREA

PHOTO 1

PHOTO 2. Northeastern View of Erosion Bankline. A County Road and State Irrigation Pipeline Parallel This Bankline (Photo Taken July 1979)

PHOTO 3. West View of Project Site at Side Channel Prior to Construction. Alignment of the Sand-Fill Dike Crosses the Side Channel at Center of Photo (Photo Taken 2 November 1980)

RIVER ROAD AREA

PHOTOS 2 AND 3

PHOTO 4. Construction of Central Gravel Core of Sand-Fill Dike for Closure of Side Channel (Photo Taken 6 November 1980)

PHOTO 5. Closure Structure Completed (Photo Taken 7 November 1980)

RIVER ROAD AREA

PHOTOS 4 AND 5

PHOTO 6. Construction Operations Showing Placement of Sand-Fill on Dike (Photo Taken 8 November 1980)

PHOTO 7. Borrow Operations of Island Looking Downstream from Hilltop (Photo Taken 11 November 1980)

RIVER ROAD AREA

PHOTOS 6 AND 7

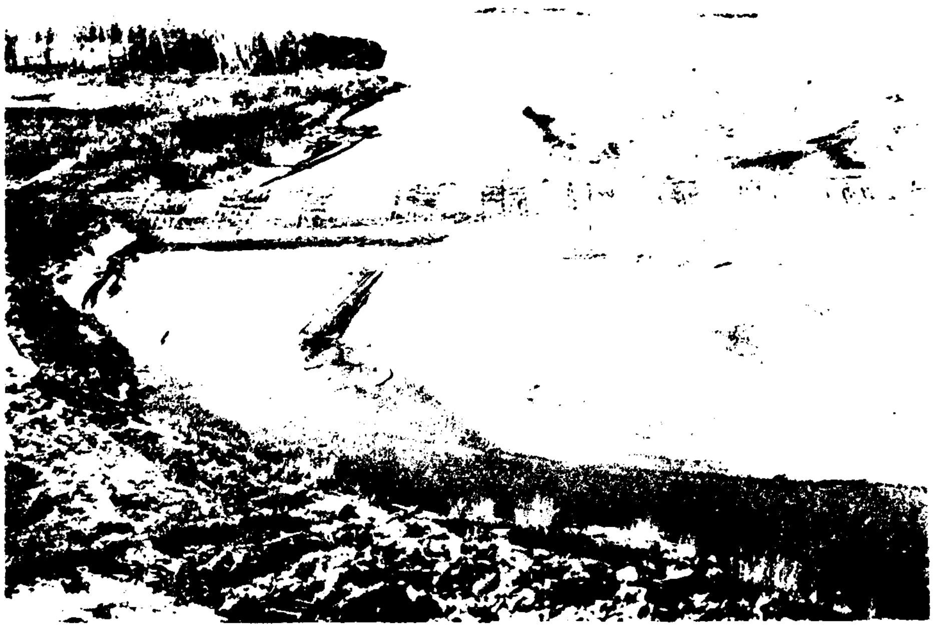


PHOTO 8. Completed Sand Fill Dike Showing Furrows Where Sprigs of Bullrushes are planted (Photo Taken 17 November 1980)

PHOTO 9. Southeastern Aerial View of Completed Sand-Fill Dike (Photo Taken 26 November 1980)

RIVER ROAD AREA

PHOTOS 8 AND 9

PHOTO 10. Eastern Aerial View of Erosion Site Before Construction
(Photo Taken 22 November 1980)

CHENEY CREEK AREA

PHOTO 10

PHOTO 11. Erosion Site Prior to Construction
Looking Downstream From Mid-Point
(Photo Taken 6 August 1980)

PHOTO 12. Erosion Site Prior to Construction
Looking Upstream From Mid-Point
(Photo Taken 6 August 1980)

CHENEY CREEK AREA

PHOTOS 11 AND 12

PHOTO 13. Erosion Site Prior to Construction Looking Downstream From Upstream End of Project (Photo Taken 6 August 1980)

PHOTO 14. Erosion Site Prior to Construction Looking Upstream From Downstream End of Project at Portable Irrigation Intake Platform (Photo Taken 22 September 1980)

CHENEY CREEK AREA

PHOTOS 13 AND 14

PHOTO 15. Field Stone Gradation Test 200# Maximum Size. Stone Used on Revetment Crown, Tiebacks and Drains (Photo Taken 13 October 1980)

PHOTO 16. Field Stone Gradation Test 500# Maximum Size. Stone Used in Revetment Toe. Both 200# and 500# Gradations Collected 25 Miles Northwest of Site. (Photo Taken 15 October 1980)

CHENEY CREEK AREA

PHOTOS 15 AND 16

PHOTO 17. Five Months After Construction Looking Downstream From Near Mid-Point of Project (Photo Taken 7 May 1981)

PHOTO 18. Five Months After Construction Looking Upstream From Near Mid-Point of Project - Note Slope Failure of Fill (Photo Taken 7 May 1981)

CHENEY CREEK AREA

PHOTOS 17 AND 18

PHOTO 19. After Construction Looking Downstream From Upstream End of Project (Photo Taken 7 May 1981)

PHOTO 20. After Construction Looking Upstream From Downstream End of Project (Photo Taken 7 May 1981)

CHENEY CREEK AREA

PHOTOS 19 AND 20

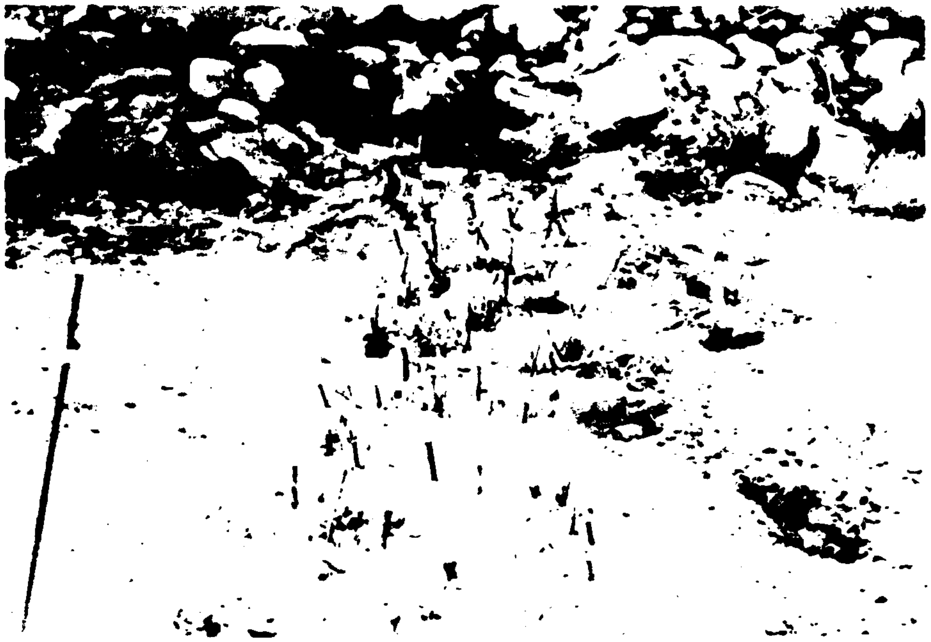


PHOTO 21. Type IV Vegetation Tie-Back Consisting of Facines and Wattles - Note Wattles Protruding Through Soil (Photo Taken 7 May 1981)

PHOTO 22. Type II Riprap Tie-Back - Note Sprouting of Vegetation Between Riprap (Photo Taken 7 May 1981)

CHENEY CREEK AREA

PHOTOS 21 AND 22

PHOTO 23. Eastern Aerial View After Construction
(Photo Taken 23 November 1980)

CHENEY CREEK AREA

PHOTO 23

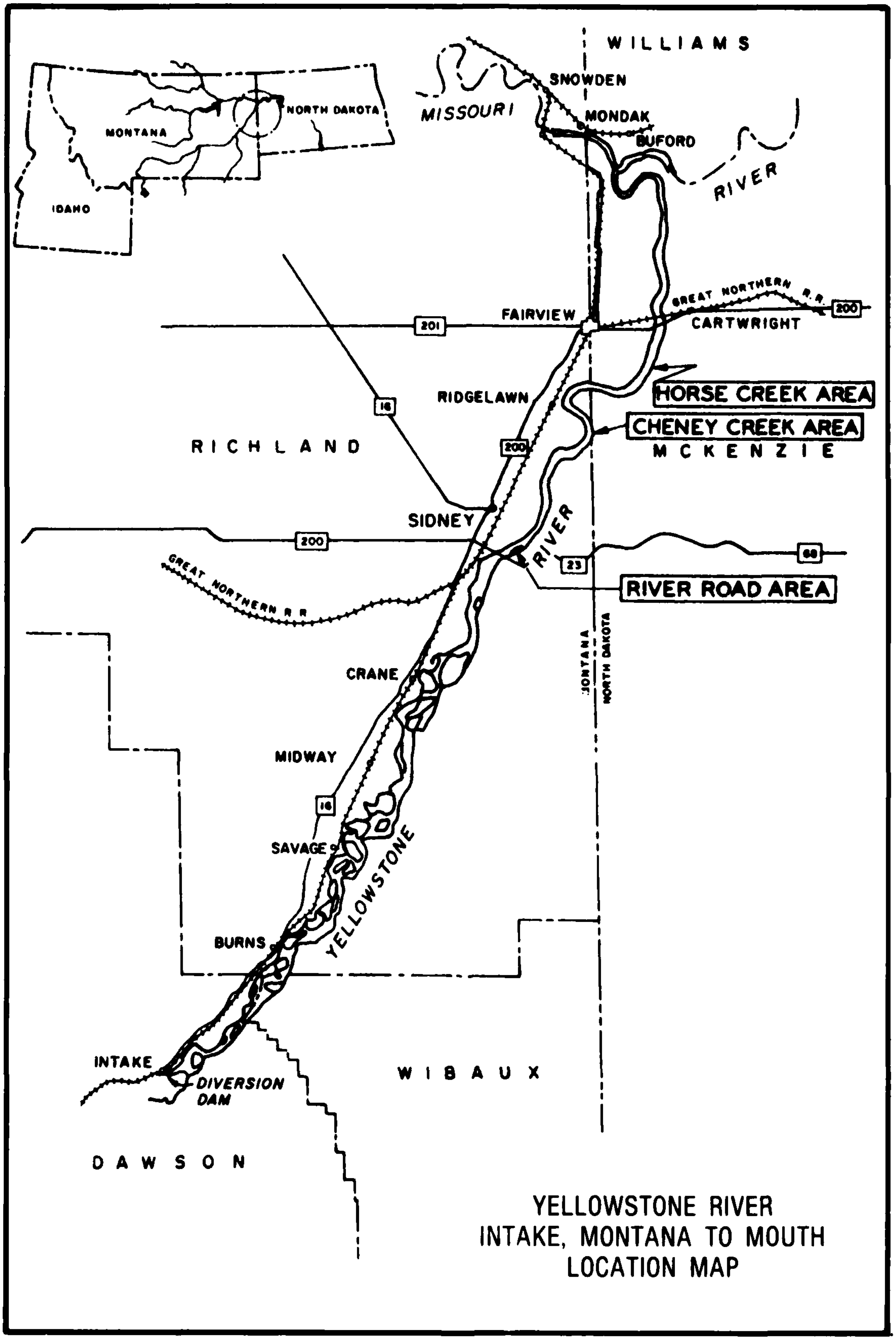

WILLIAMS
SNOWDEN
MISSOURI
MONDAK
BUFORD
RIVER
NORTH DAKOTA
MONTANA
IDAHO
GREAT NORTHERN R.R.
200
FAIRVIEW
201
CARTWRIGHT
HORSE CREEK AREA
16
RIDGELAWN
CHENEY CREEK AREA
RICHLAND
200
MCKENZIE
SIDNEY
RIVER
200
23
68
GREAT NORTHERN R.R.
RIVER ROAD AREA
MONTANA
NORTH DAKOTA
CRANE
MIDWAY
16
SAVAGE
YELLOWSTONE
BURNS
INTAKE
DIVERSION
DAM
WIBAUX
DAWSON
YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
LOCATION MAP

PLATE 1

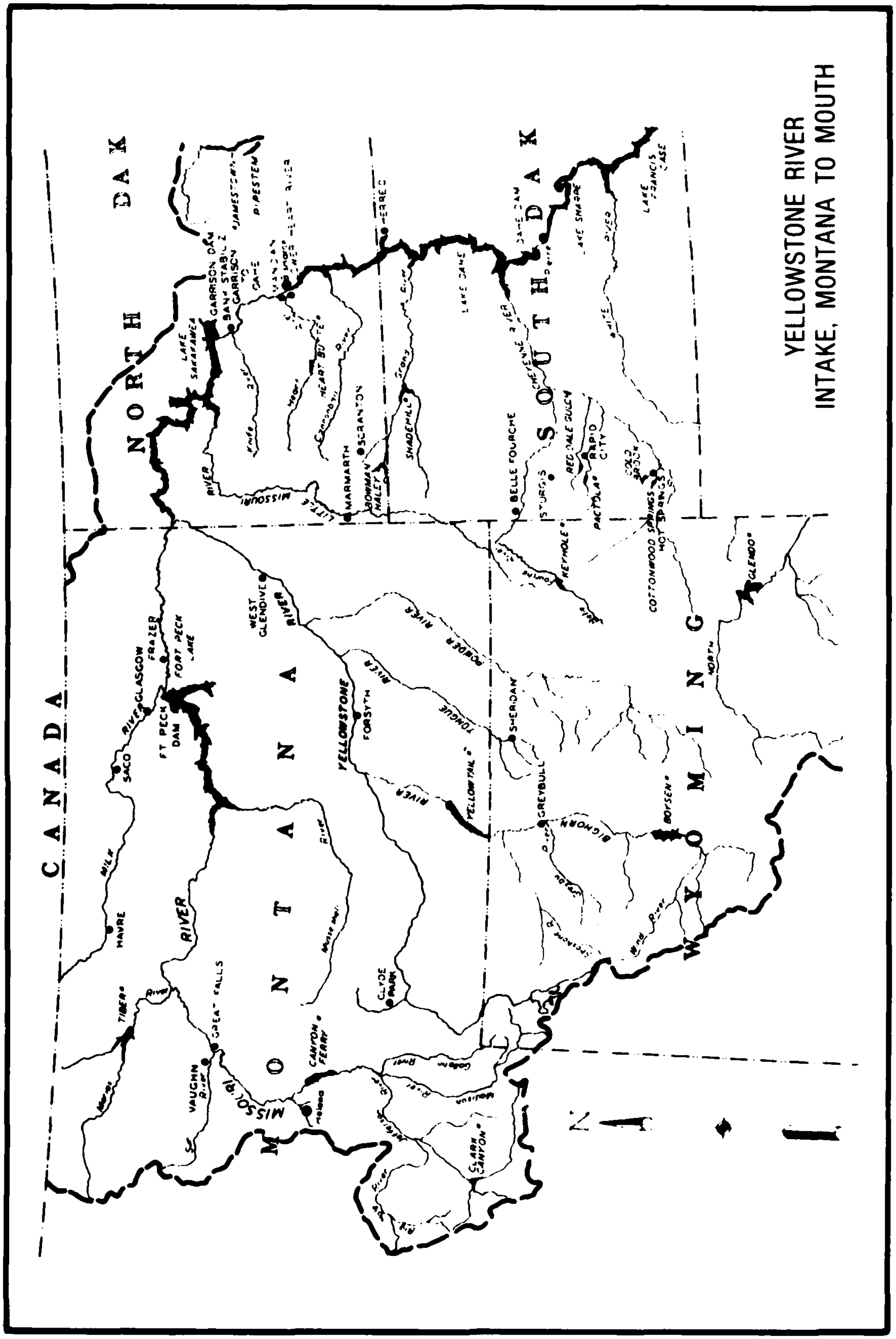
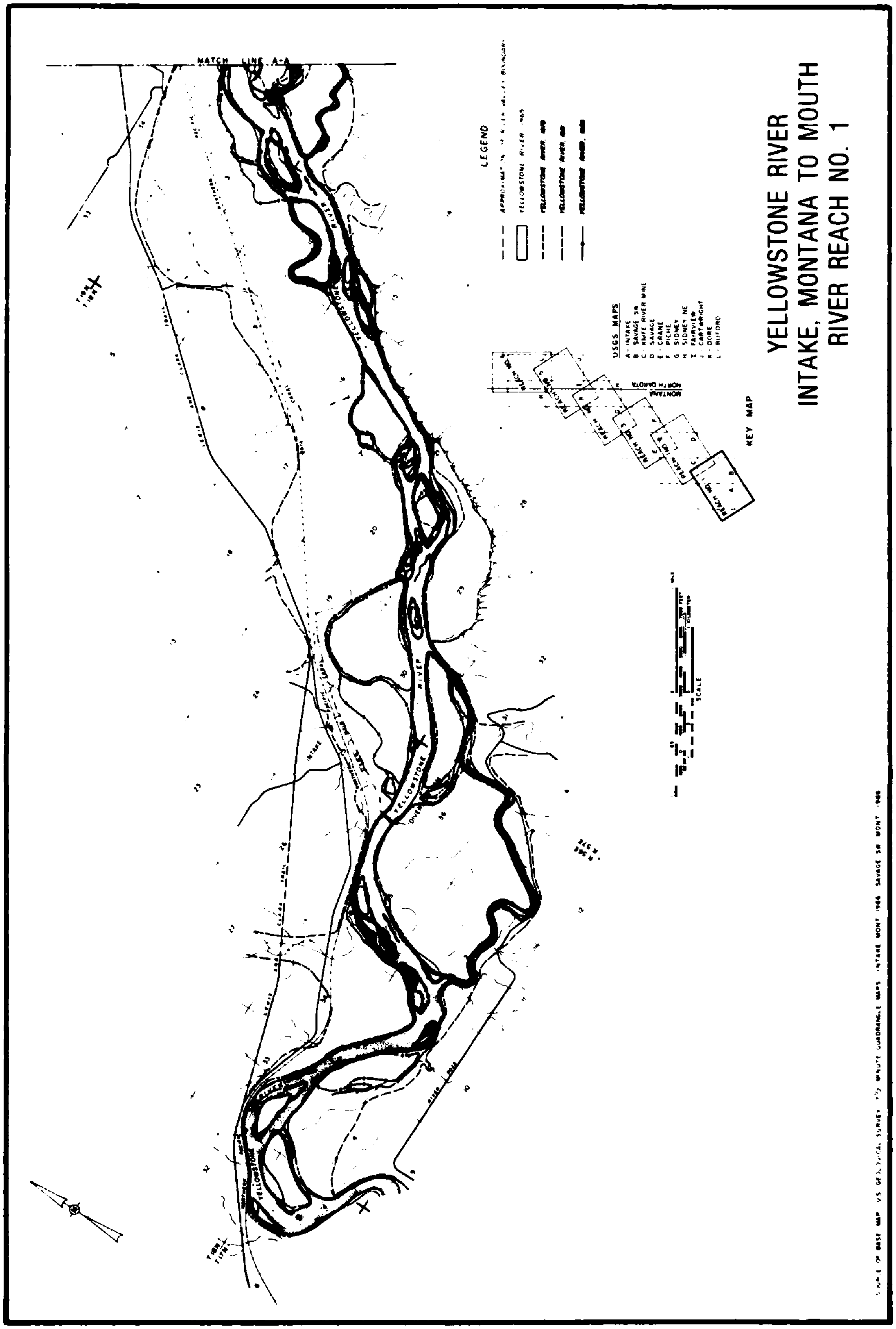

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CANADA
NORTH DAK
SOUTH DAK
MONTANA
WYOMING
GARRISON DAM
LAKE SAKAKAWEA
FT PECK DAM
FORT PECK LAKE
GLASGOW
FRAZER
SACO
HAVRE
TIBER
GREAT FALLS
CANYON FERRY
CLARK CANYON
CLYDE PARK
YELLOWTAIL
FORSYTH
WEST GLENDIVE
YELLOWSTONE
RIVER
MISSOURI
TONGUE RIVER
POWDER RIVER
SHERIDAN
GREYBULL
BOYSEN
GLENDO
KEYHOLE
BELLE FOURCHE
STURGIS
RAPID CITY
PACTOLA
COTTONWOOD SPRINGS
HOT SPRINGS
SHADEHILL
MARMARTH
BOWMAN HALEY
SCRANTON
HEART BUTTE
LAKE FRANCIS CASE
LAKE OAHE
CHEYENNE RIVER
VAUGHN
MILK
BIGHORN
WIND RIVER

PLATE 2



MATCH LINE A-A
LEGEND
USGS MAPS
NORTH DAKOTA
MONTANA
KEY MAP
YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 1

PLATE 3

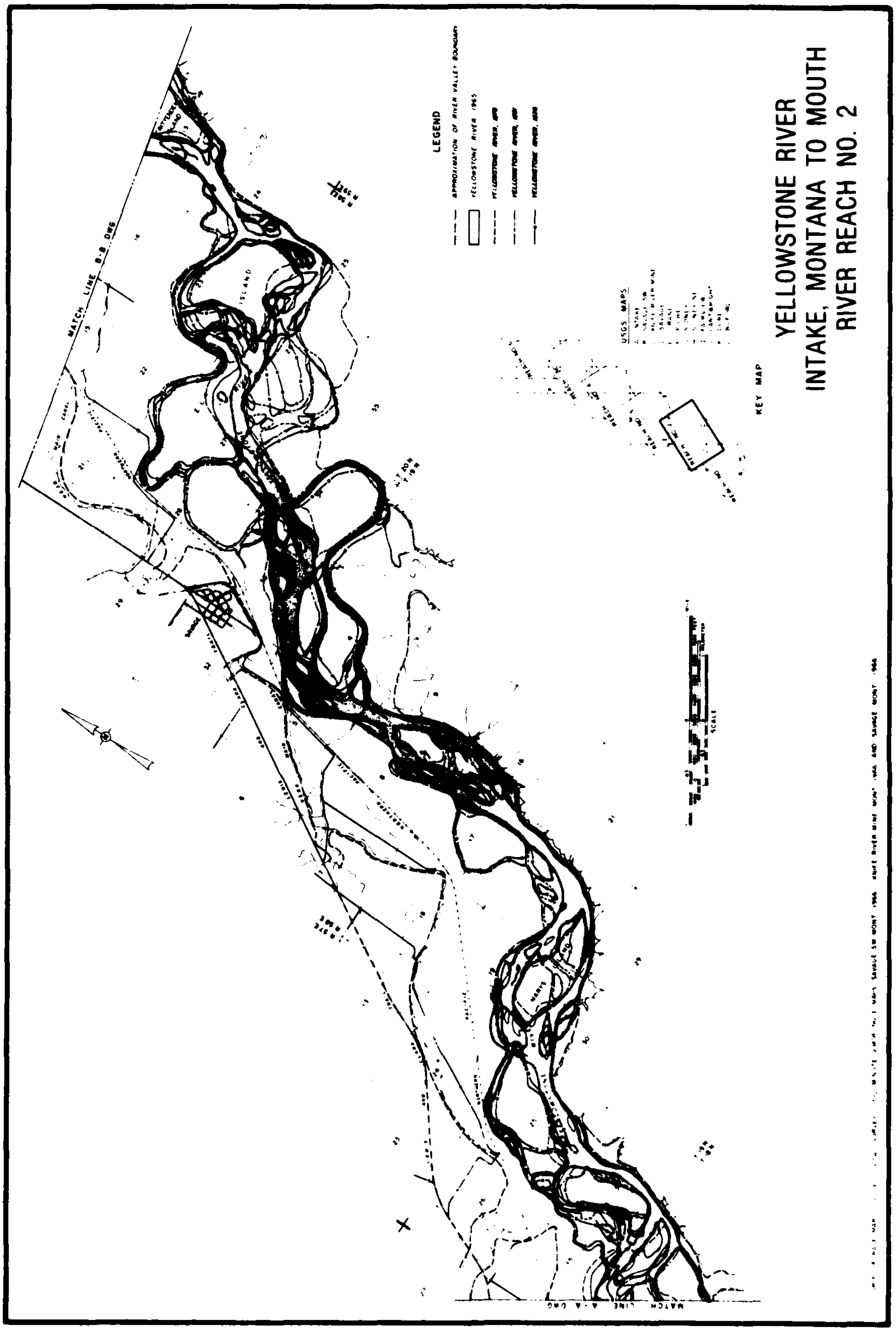

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 2
LEGEND
KEY MAP
USGS MAPS
SCALE
MATCH LINE B-B DWG
MATCH LINE A-A DWG
ISLAND

PLATE 4

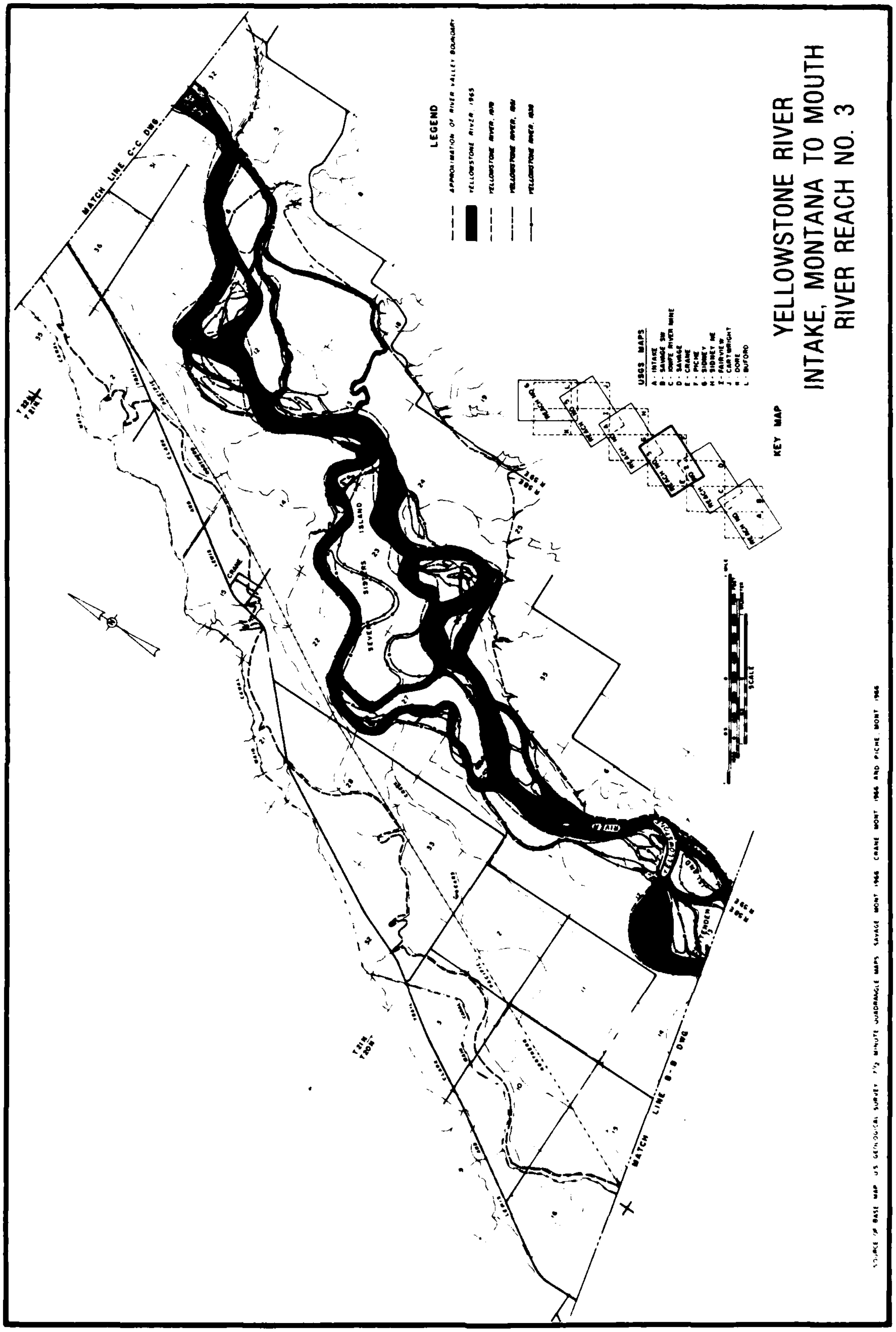

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 3
LEGEND
KEY MAP
USGS MAPS
SCALE
MATCH LINE C-C DWG
MATCH LINE B-B DWG

PLATE 5

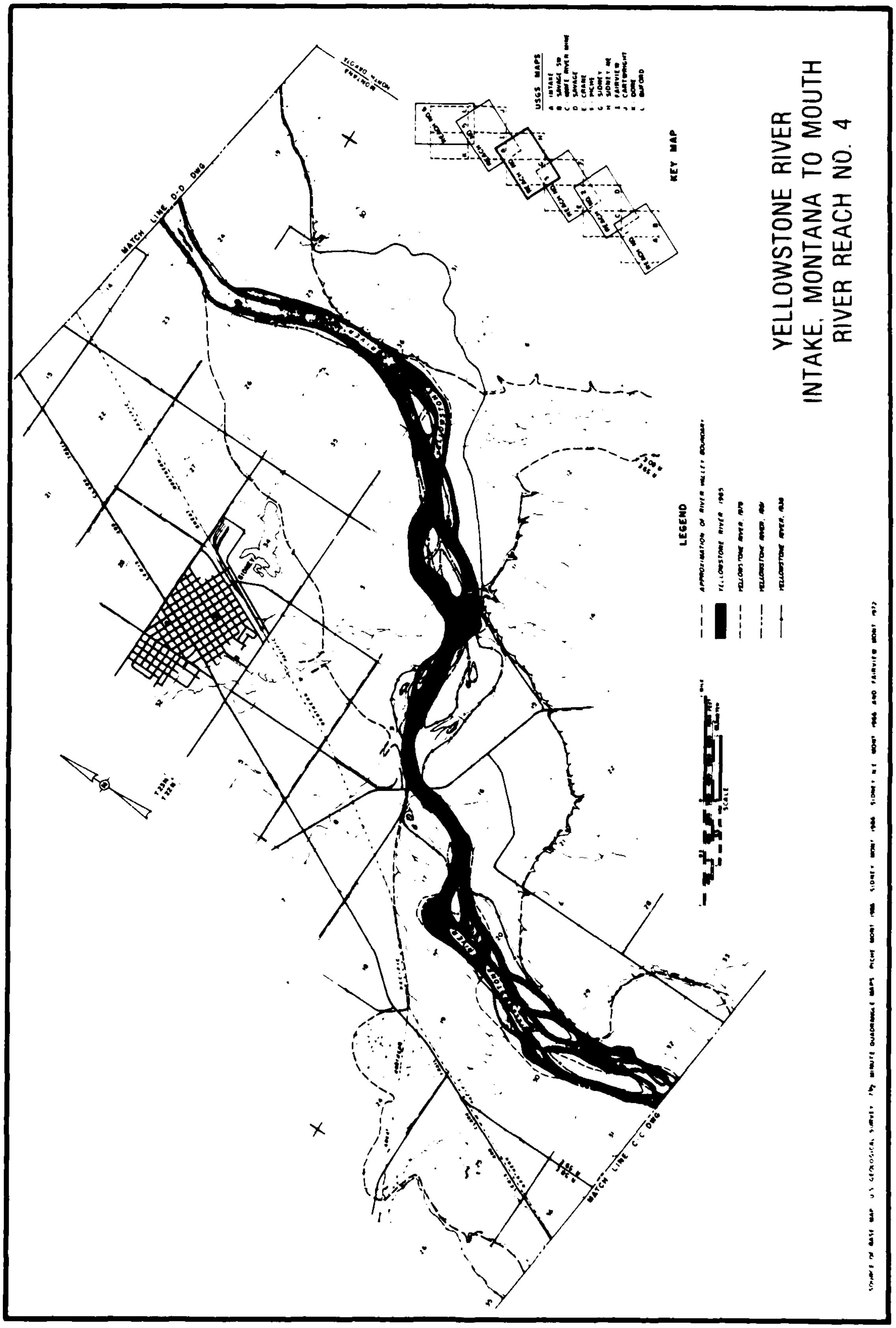

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 4
LEGEND
KEY MAP
USGS MAPS
MATCH LINE D-D DWG
MATCH LINE C-C DWG

PLATE 6

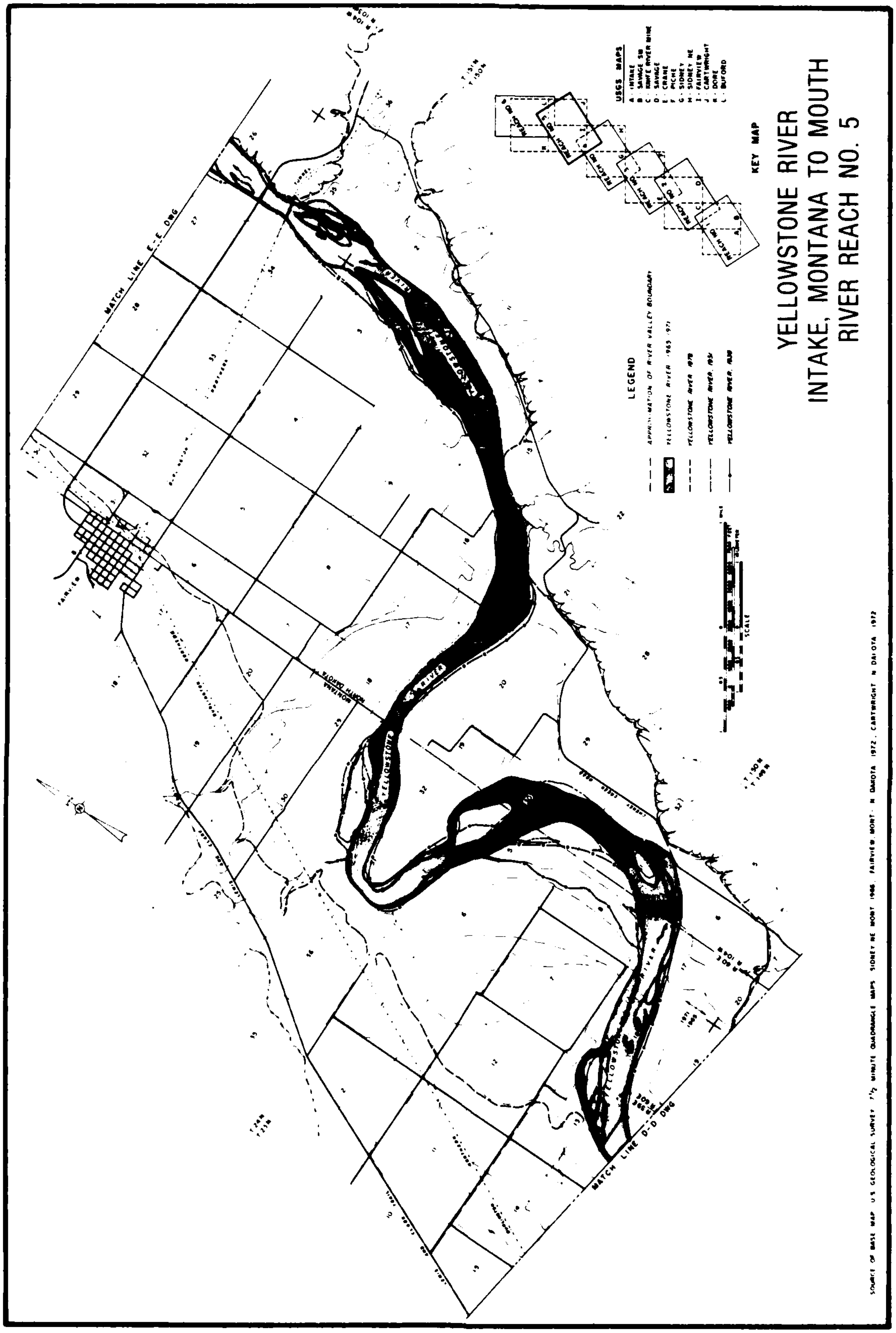

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 5
KEY MAP
USGS MAPS
LEGEND
SCALE
MATCH LINE E-E DWG
MATCH LINE D-D DWG
YELLOWSTONE RIVER
MONTANA
NORTH DAKOTA

PLATE 7

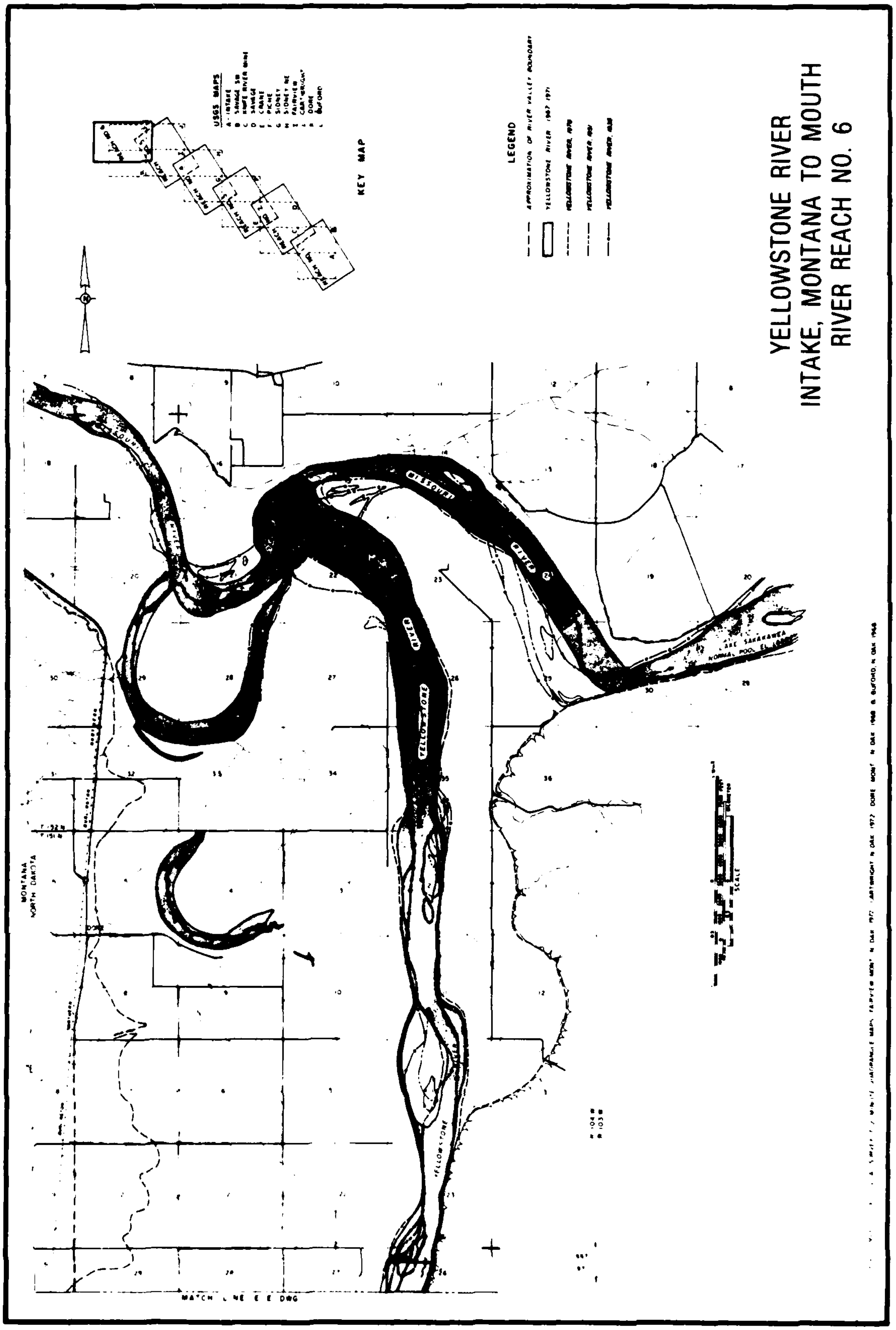

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER REACH NO. 6
KEY MAP
LEGEND
USGS MAPS
MONTANA
NORTH DAKOTA
SCALE

PLATE 8

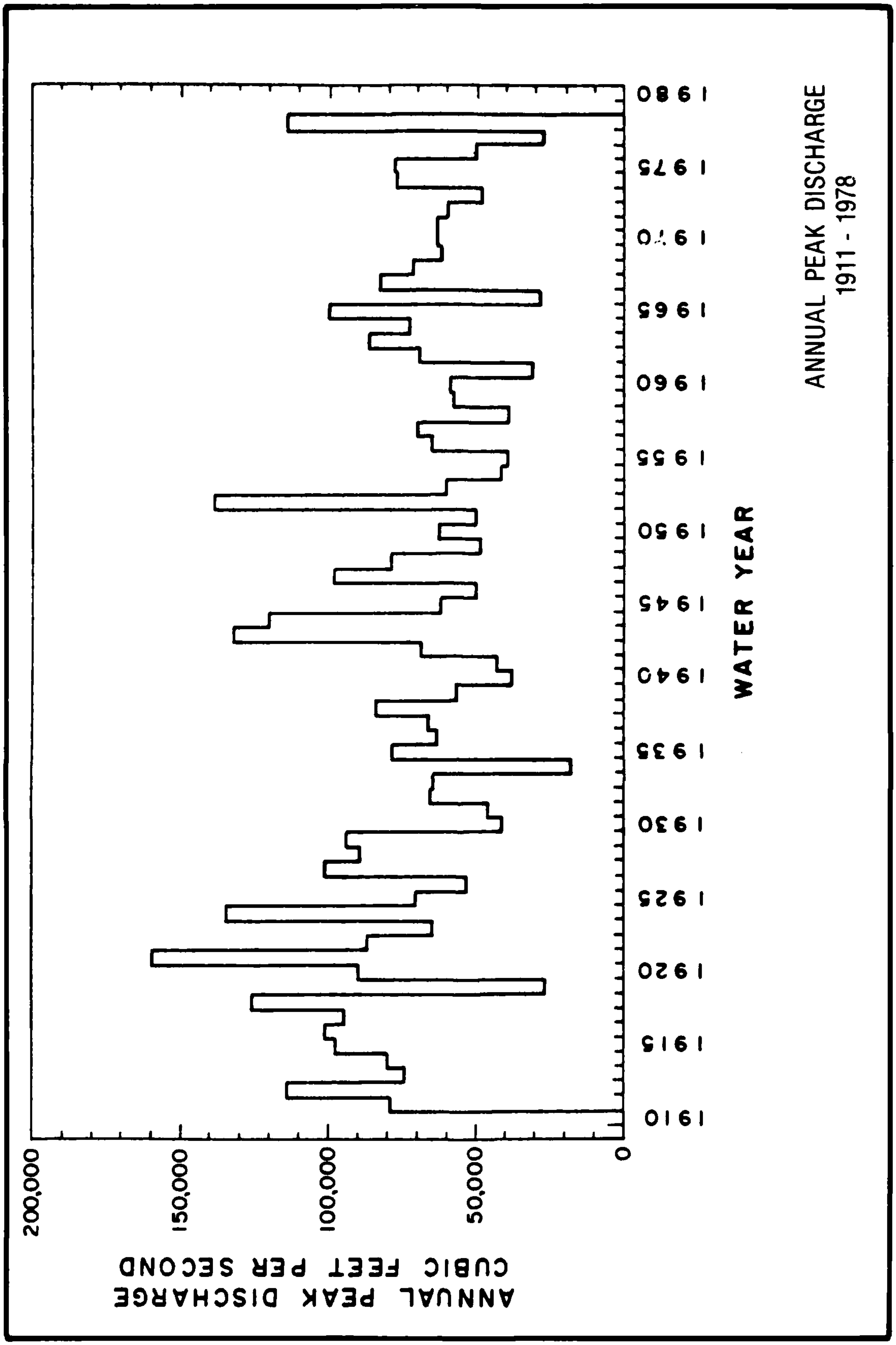
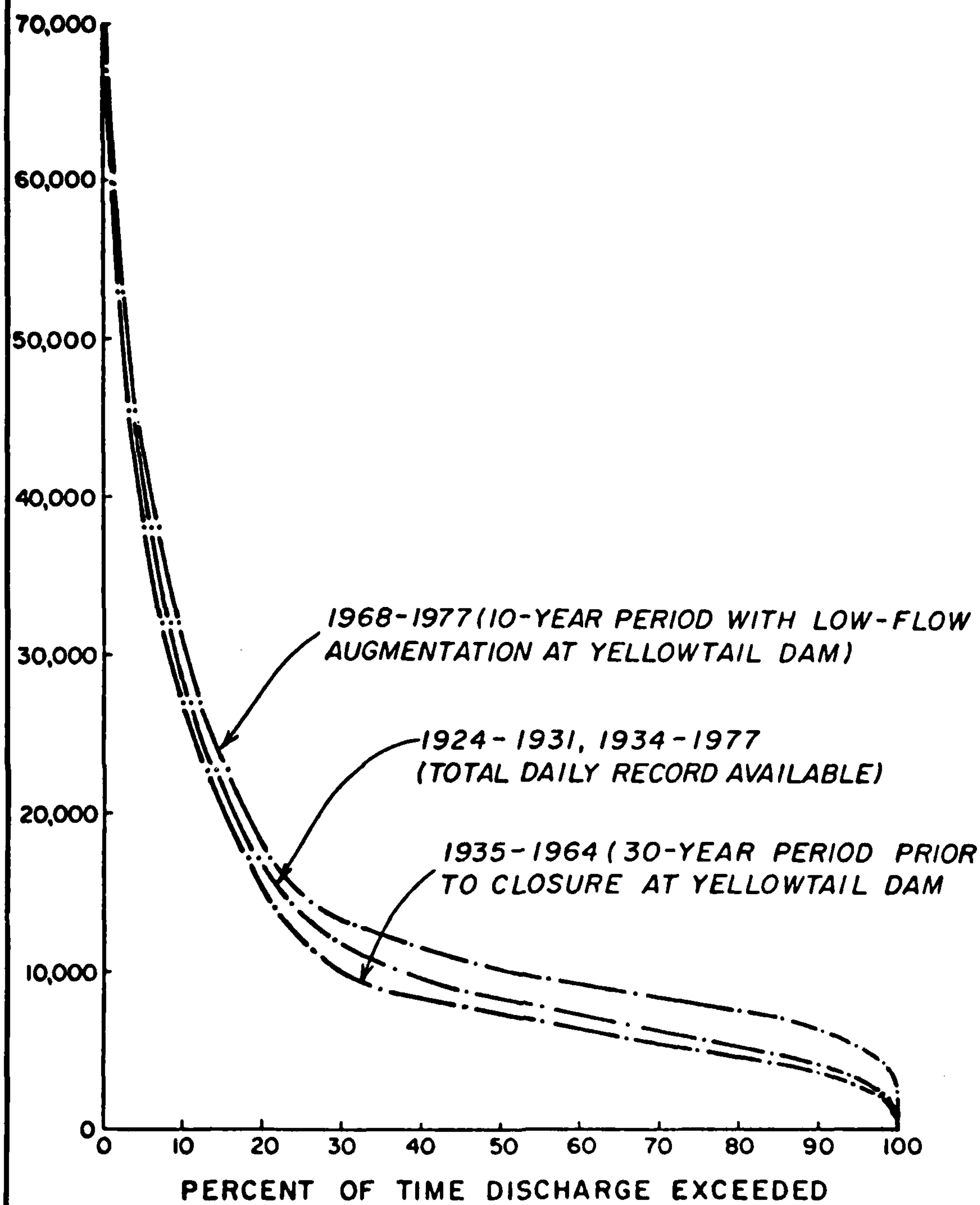
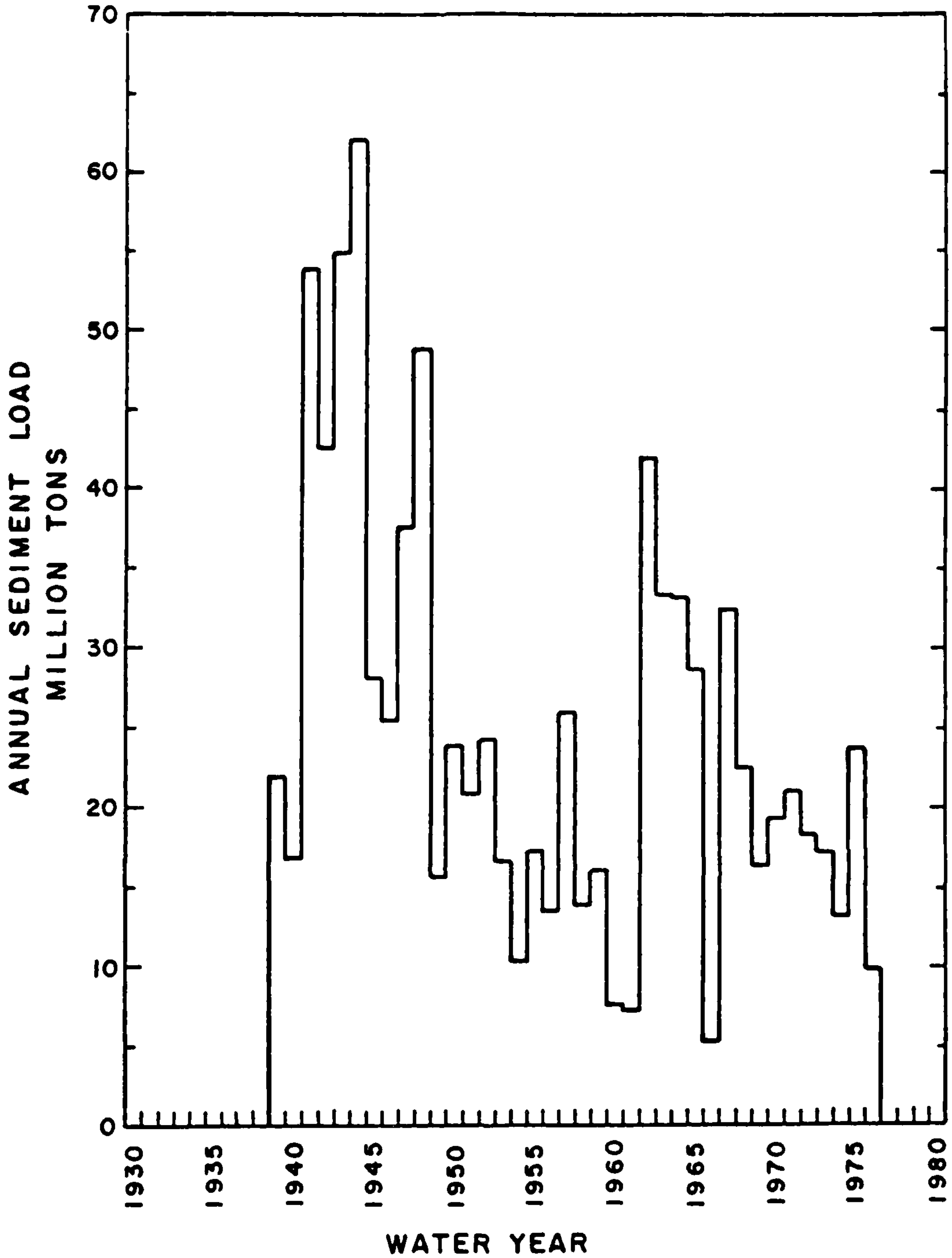

ANNUAL PEAK DISCHARGE
1911 - 1978
ANNUAL PEAK DISCHARGE
CUBIC FEET PER SECOND
WATER YEAR
0
50,000
100,000
150,000
200,000
1910
1915
1920
1925
1930
1935
1940
1945
1950
1955
1960
1965
1970
1975
1980

PLATE 9



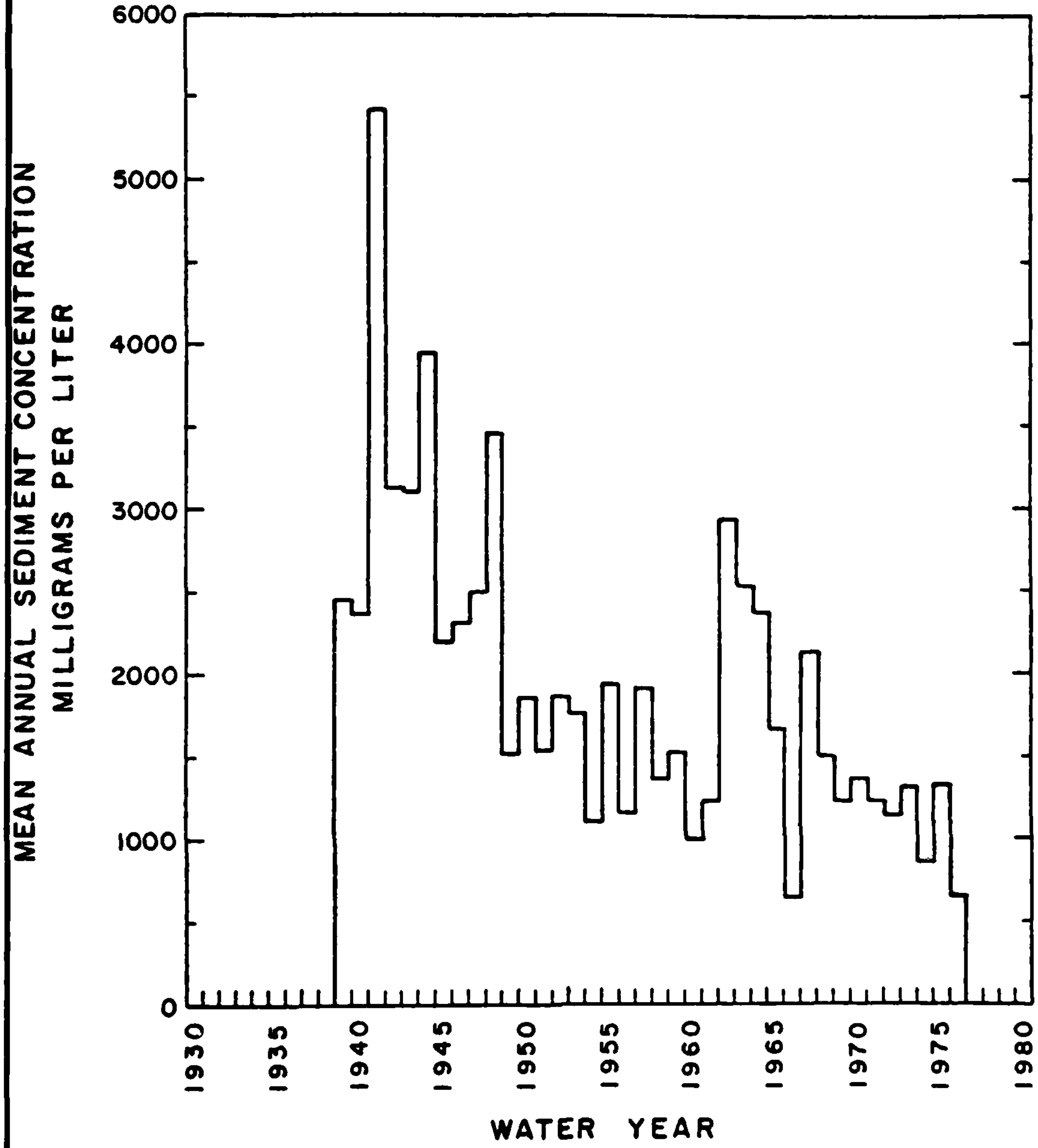
YELLOWSTONE RIVER
DAILY FLOW-DURATION CURVES

PLATE 10



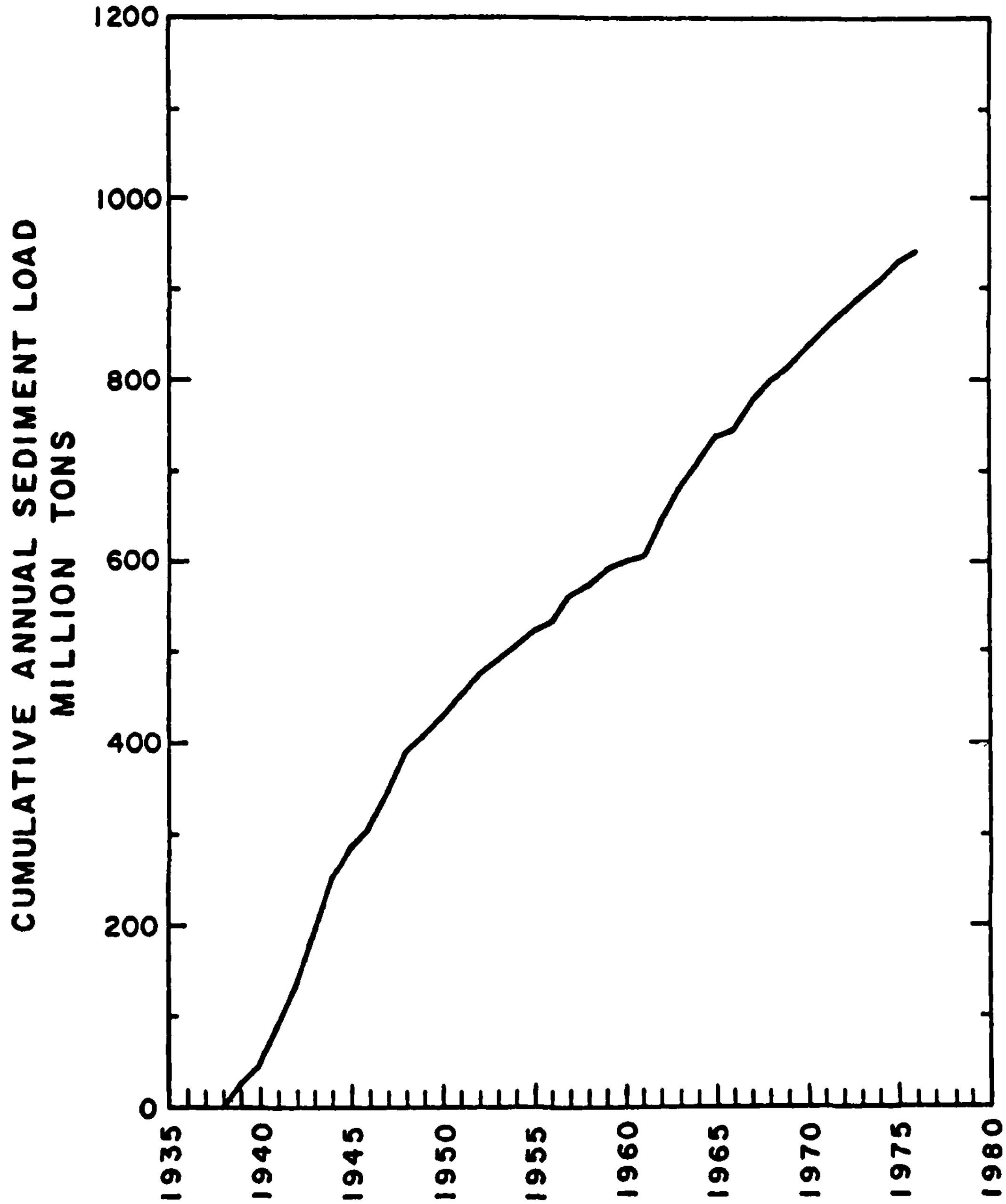
ANNUAL SEDIMENT LOAD
1939 - 1976

PLATE 11



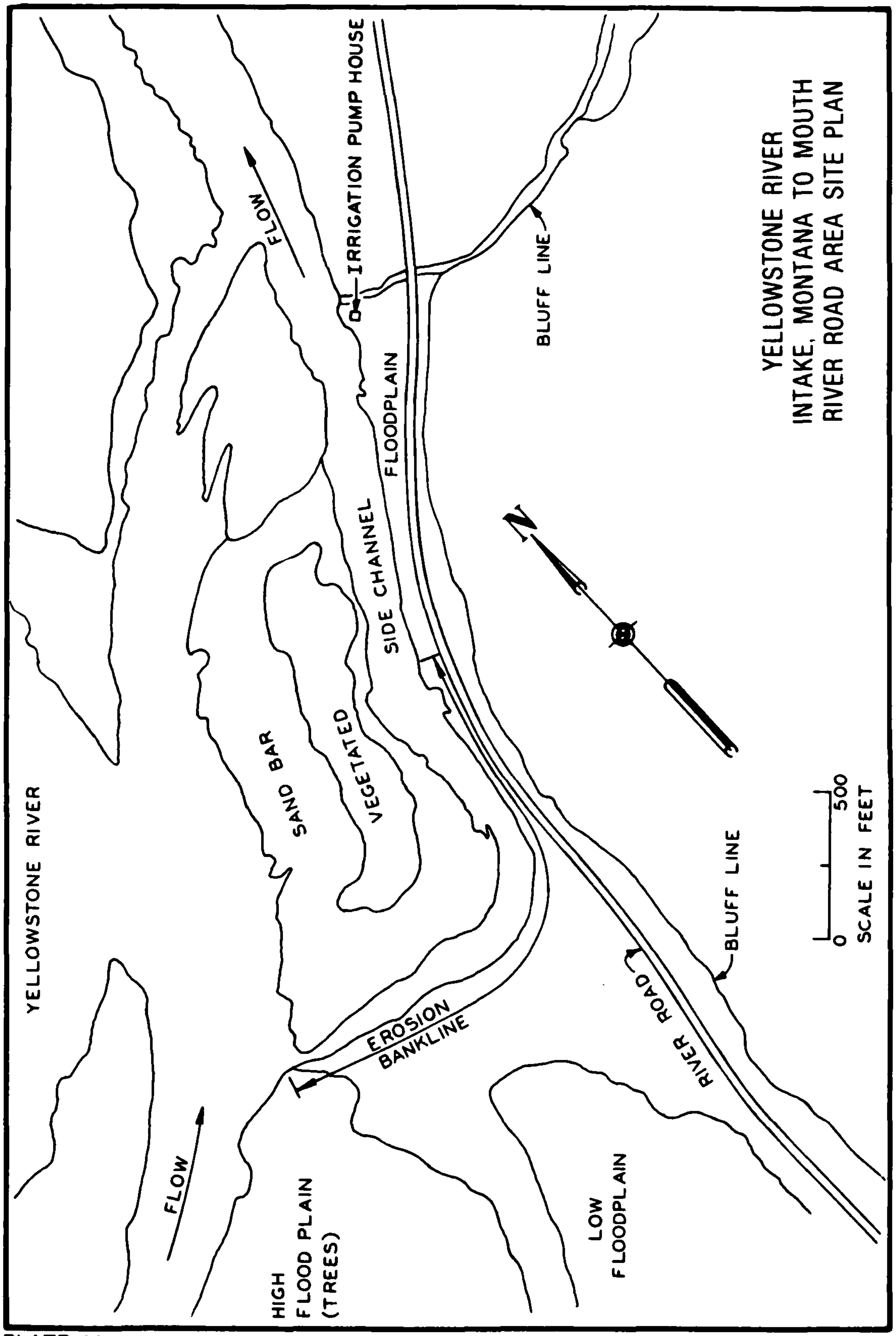
MEAN ANNUAL SEDIMENT CONCENTRATION
1939 - 1976

PLATE 12



CUMULATIVE ANNUAL SEDIMENT LOAD
1939 - 1976

PLATE 13


YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER ROAD AREA SITE PLAN
IRRIGATION PUMP HOUSE
FLOW
BLUFF LINE
FLOODPLAIN
SIDE CHANNEL
VEGETATED
SAND BAR
YELLOWSTONE RIVER
N
0
500
SCALE IN FEET
BLUFF LINE
RIVER ROAD
EROSION
BANKLINE
FLOW
HIGH
FLOOD PLAIN
(TREES)
LOW
FLOODPLAIN
PLATE 14

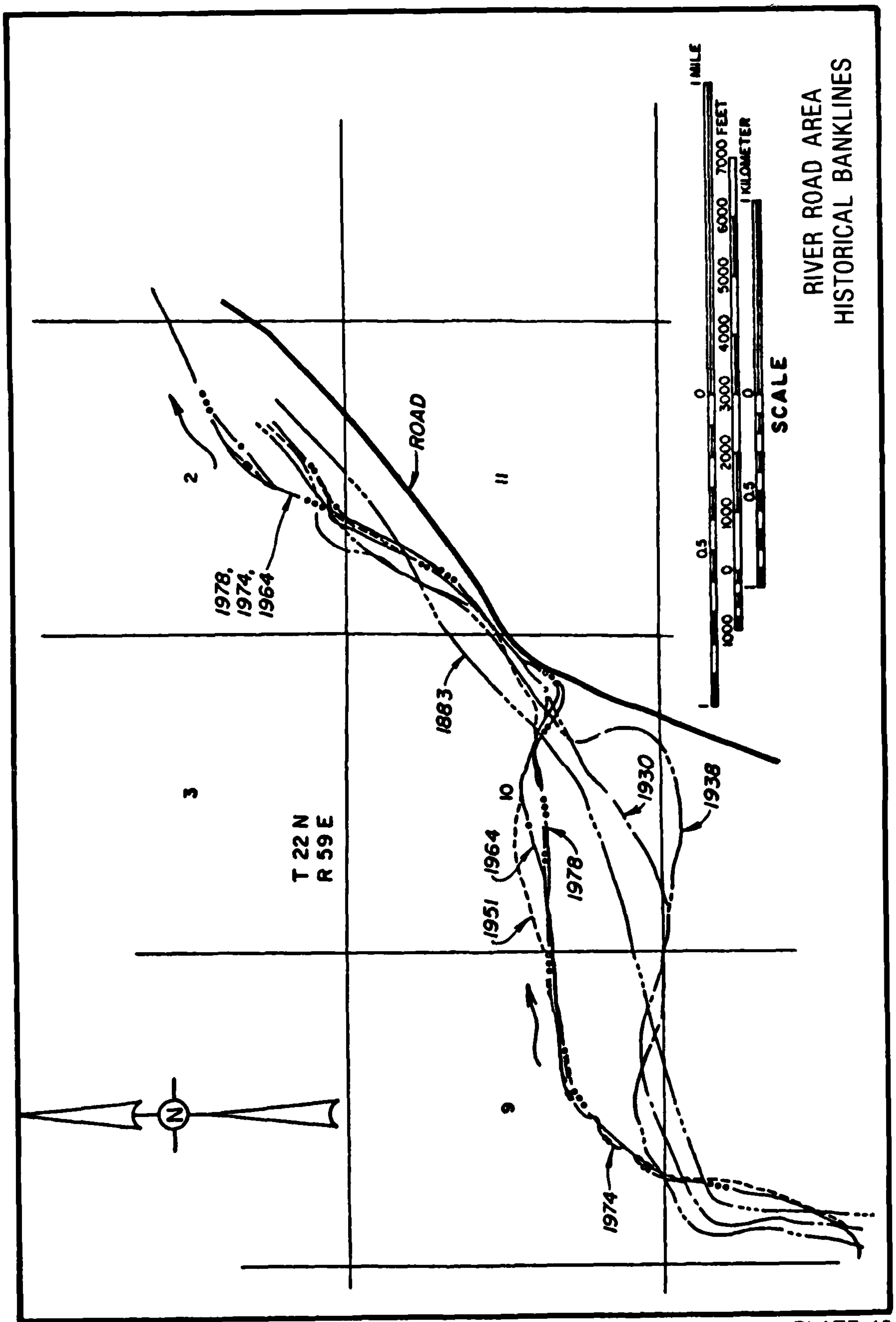

RIVER ROAD AREA
HISTORICAL BANKLINES
SCALE
1 MILE
7000 FEET
1 KILOMETER
ROAD
1978,
1974,
1964
1883
1930
1938
1964
1951
1978
1974
T 22 N
R 59 E
2
3
9
10
11

PLATE 15

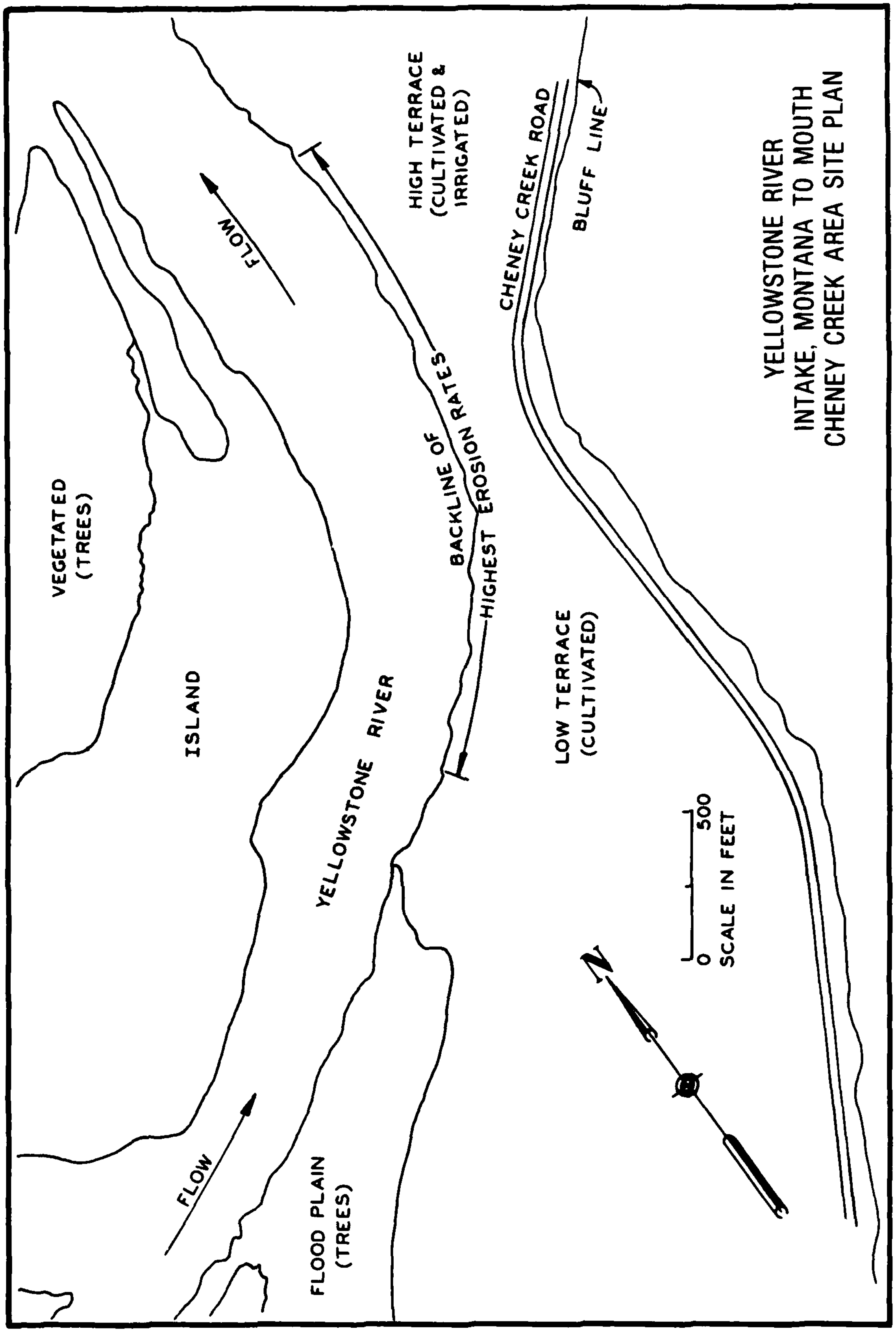
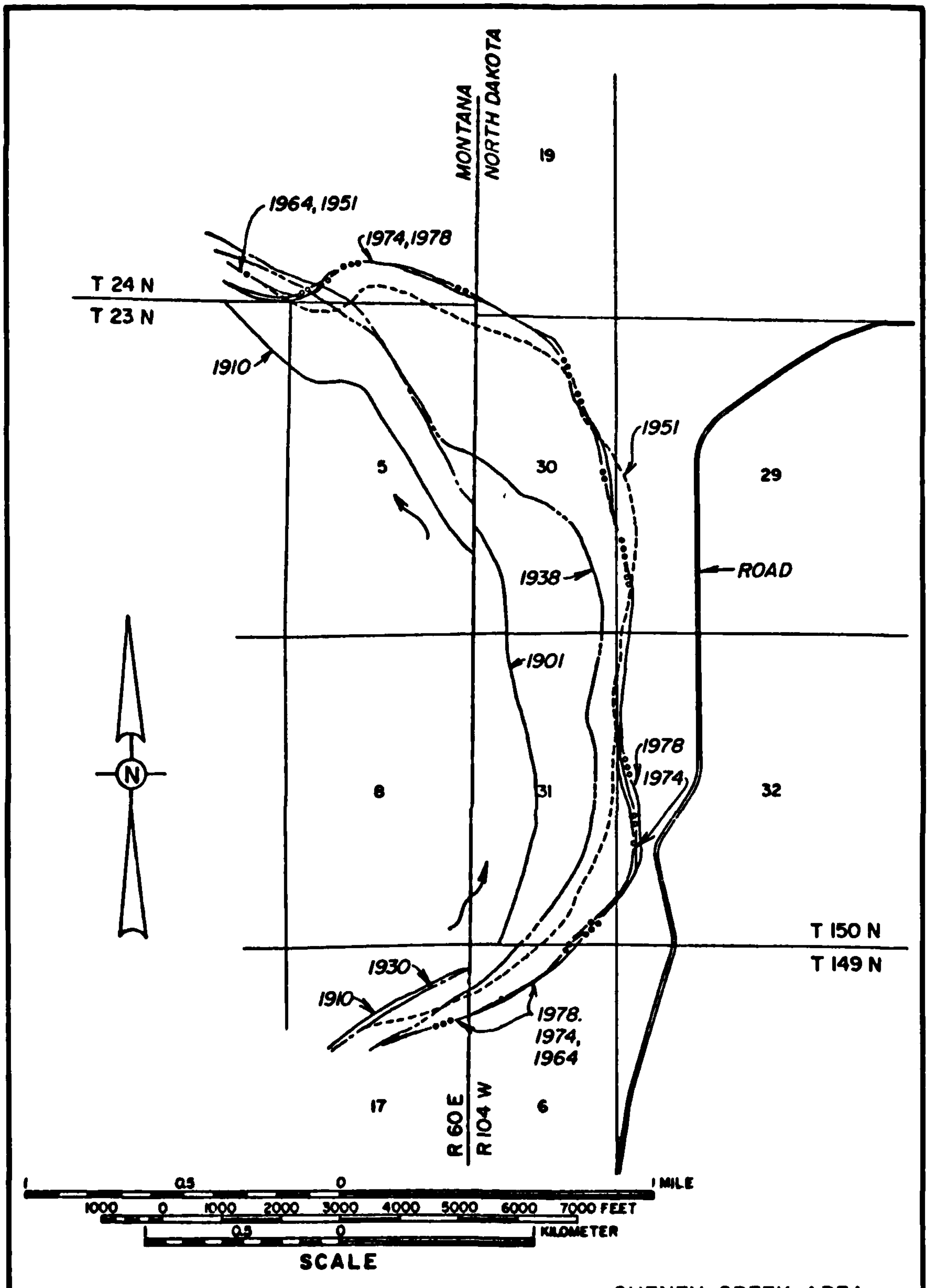

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA SITE PLAN
HIGH TERRACE
(CULTIVATED &
IRRIGATED)
CHENEY CREEK ROAD
BLUFF LINE
FLOW
BACKLINE OF
HIGHEST EROSION RATES
VEGETATED
(TREES)
ISLAND
YELLOWSTONE RIVER
LOW TERRACE
(CULTIVATED)
0
500
SCALE IN FEET
N
FLOW
FLOOD PLAIN
(TREES)

PLATE 16



CHENEY CREEK AREA
HISTORICAL BANKLINES

PLATE 17

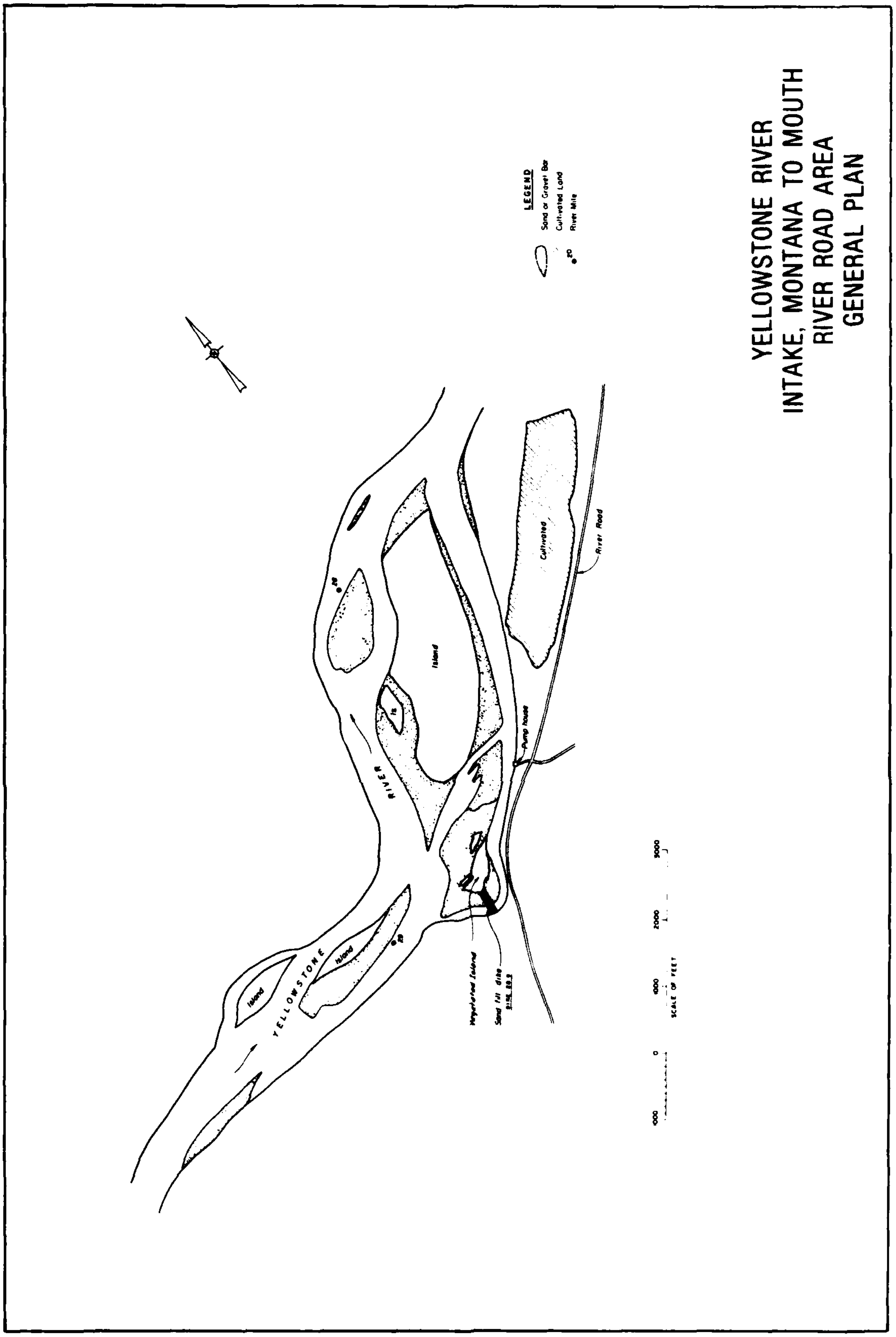
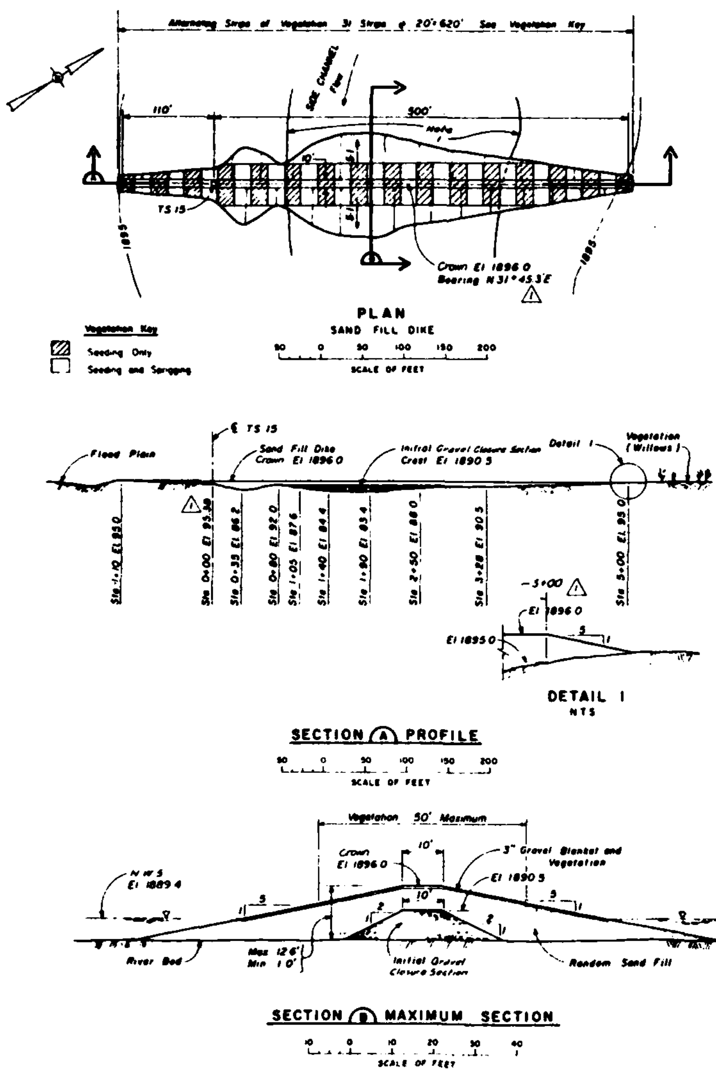
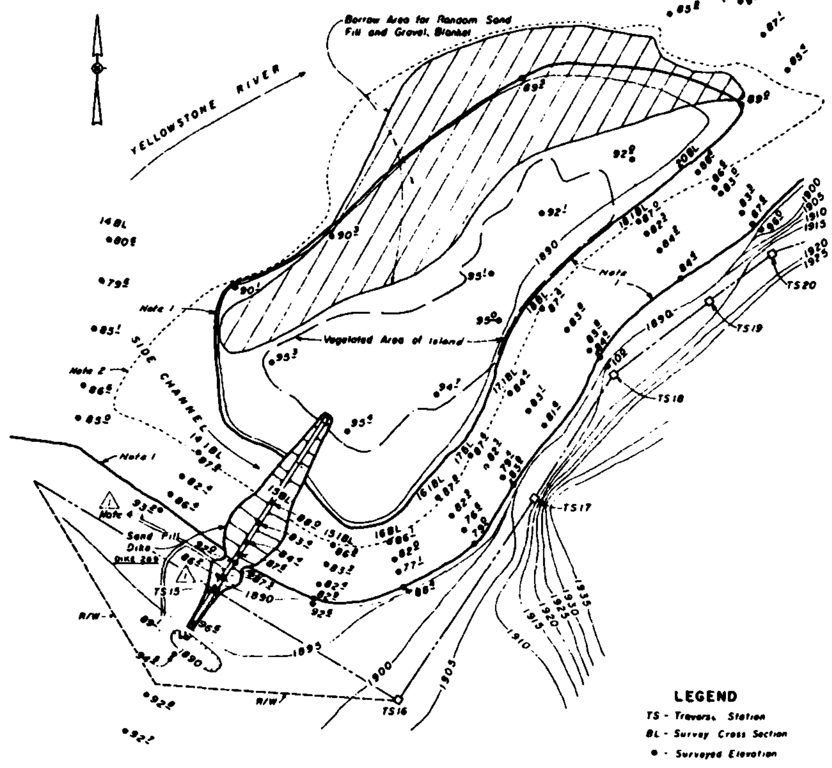

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER ROAD AREA
GENERAL PLAN
LEGEND
Sand or Gravel Bar
Cultivated Land
River Mile
Cultivated
River Road
Island
Is
Pump house
RIVER
YELLOWSTONE
Island
Island
Vegetated Island
Sand fill dike
SCALE OF FEET
1000
0
1000
2000
3000

PLATE 18





LEGEND

TS - Traverse Station

BL - Survey Cross Section

• - Surveyed Elevation

NOTES:

1. This is the water's edge taken from June 10, 1979 Survey
2. This is the approximate water's edge at elevation 1887.5 msl and represents a streamflow of approximately 12,000 cubic feet per sec. It is anticipated that the river will have this water surface elevation or less during the construction period specified where the total borrow area will be exposed. After initial closure, if the river has not receded sufficiently to expose the total borrow area, those portions of the borrow area that are exposed may be excavated as identified and approved by the Contracting Officer. The methods of borrow excavation shall be in accordance with the specifications.
3. The boundaries indicated for the borrow area and the edge of the vegetated portion of the island are based on the latest field conditions and will depend upon actual field conditions.
4. Revise the ditch entrance to the side channel to exit riverward of the sand-fill dike and such that flow is not directed towards the sand-fill dike.

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
RIVER ROAD AREA
PLANS AND SECTIONS

PLATE 19

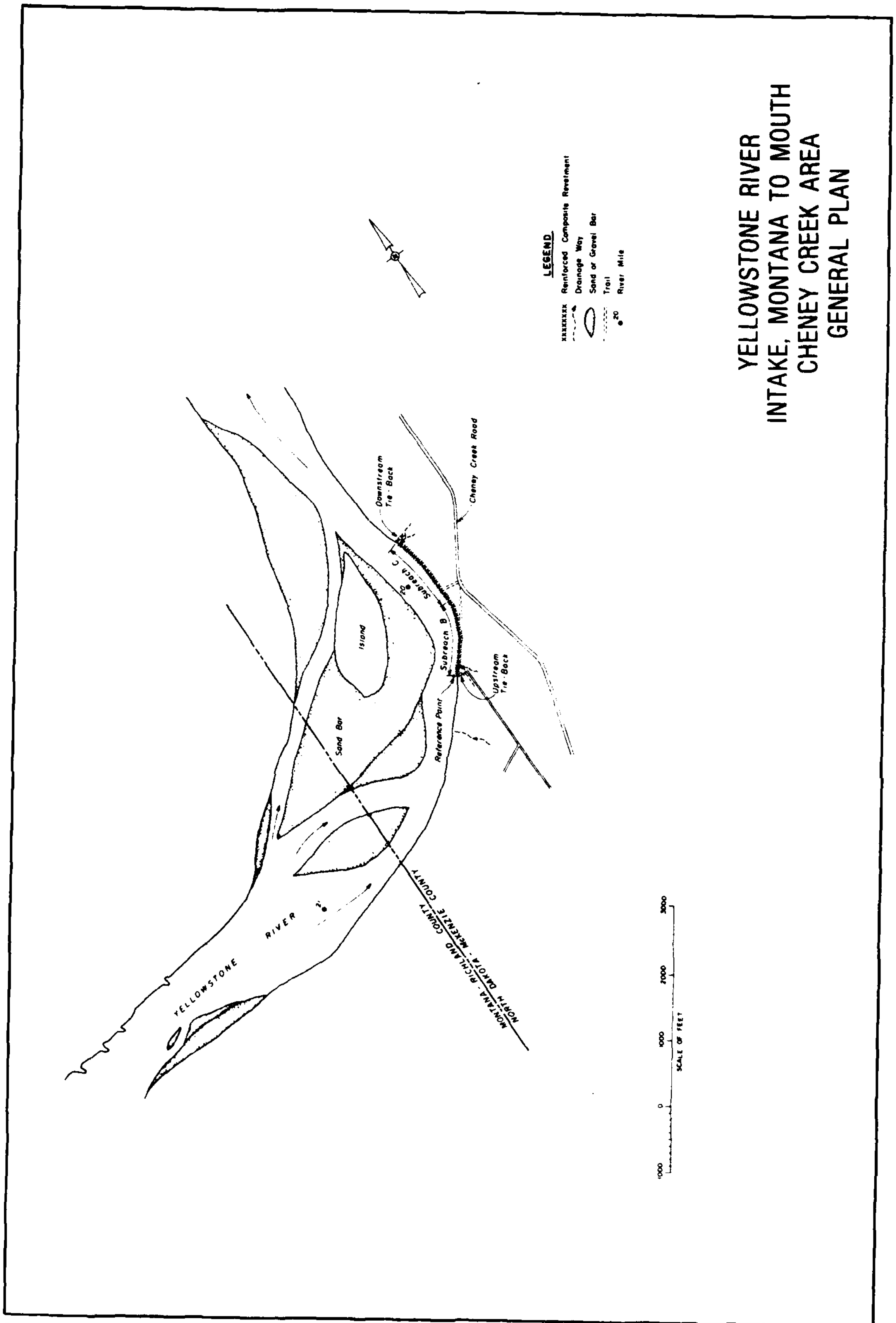
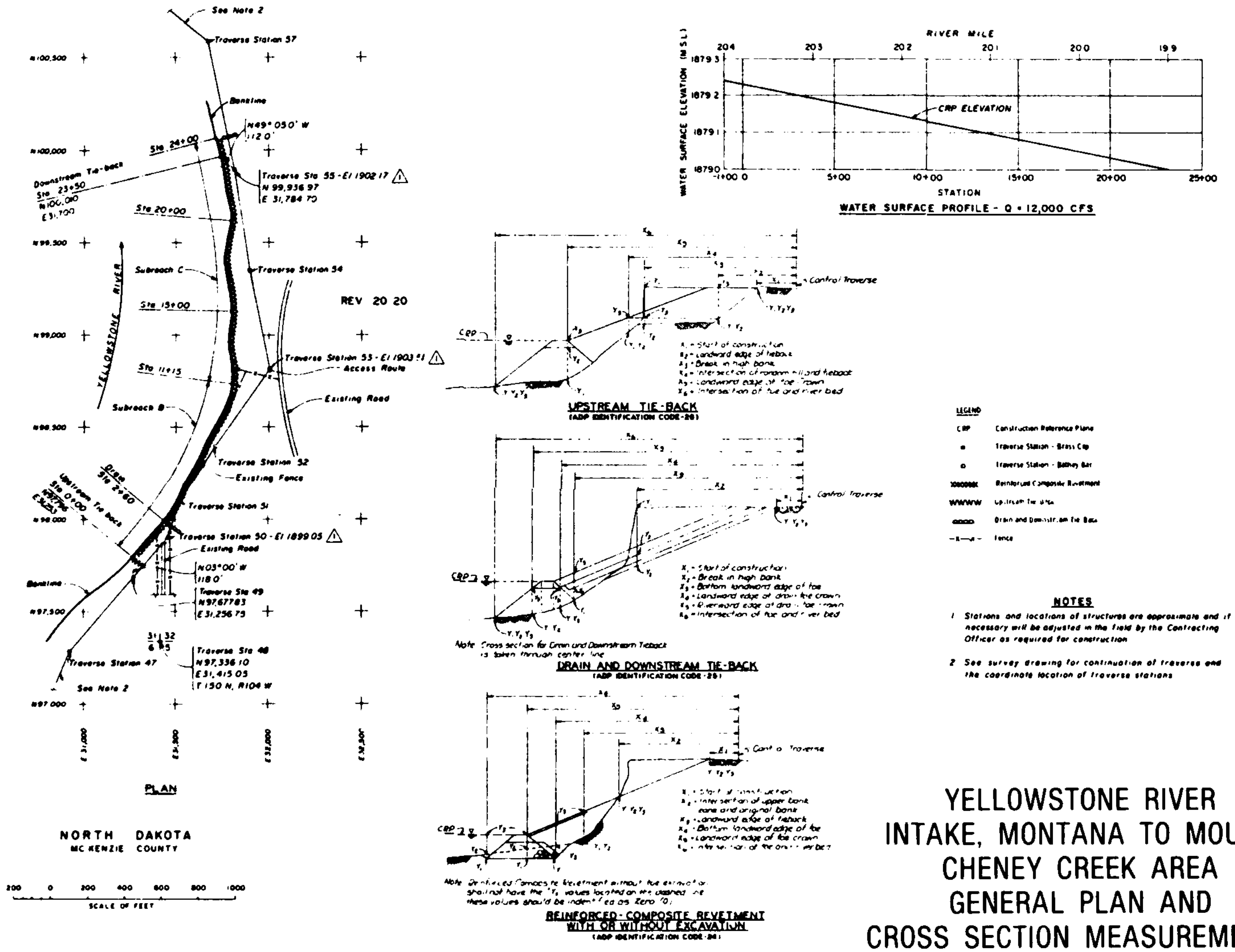
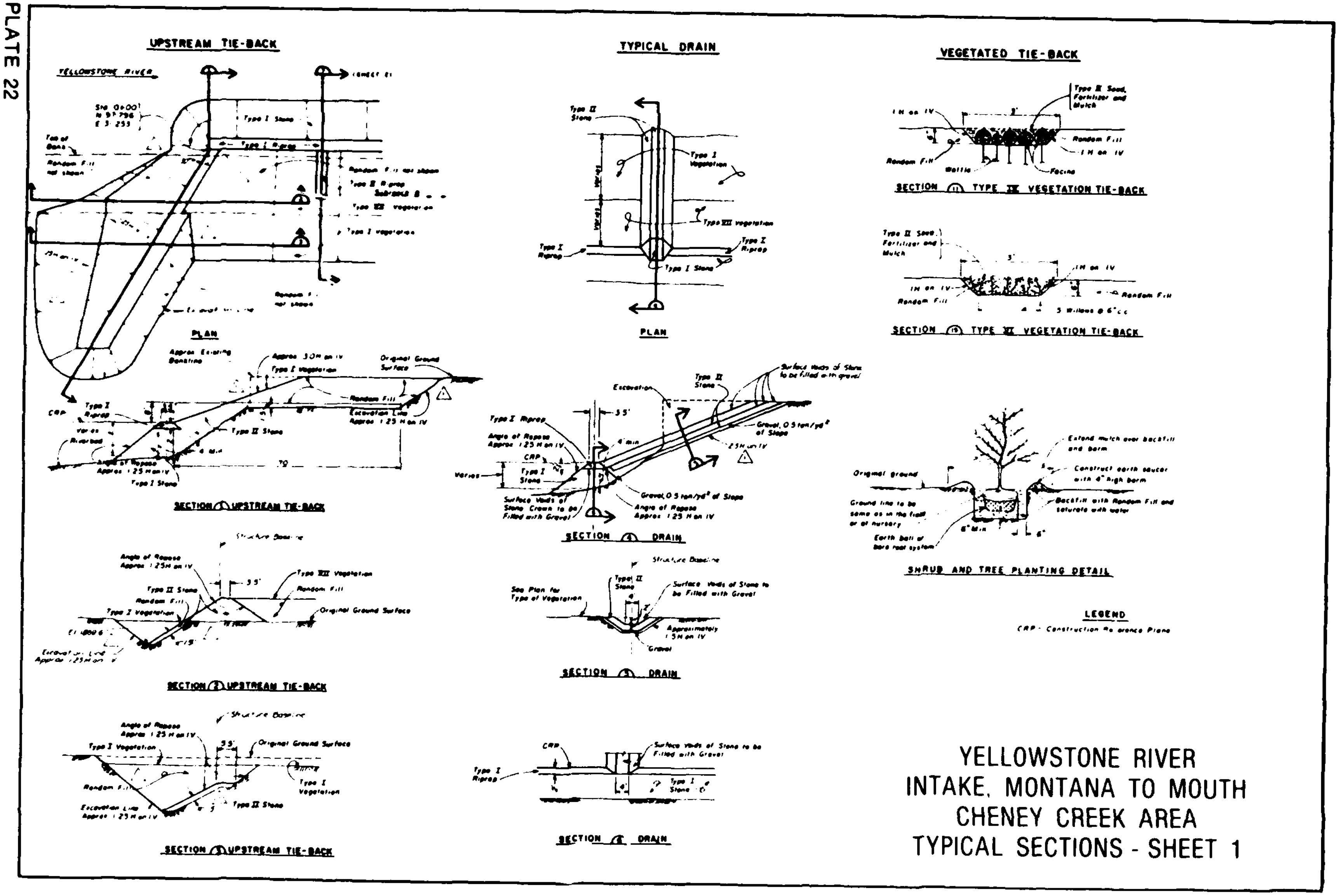

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA
GENERAL PLAN
LEGEND
Reinforced Composite Revetment
Drainage Way
Sand or Gravel Bar
Trail
River Mile
Cheney Creek Road
Downstream Tie-Back
Upstream Tie-Back
Subreach C
Subreach B
Island
Sand Bar
Reference Point
NORTH DAKOTA - McKENZIE COUNTY
MONTANA - RICHLAND COUNTY
YELLOWSTONE RIVER
1000 0 1000 2000 3000
SCALE OF FEET

PLATE 20



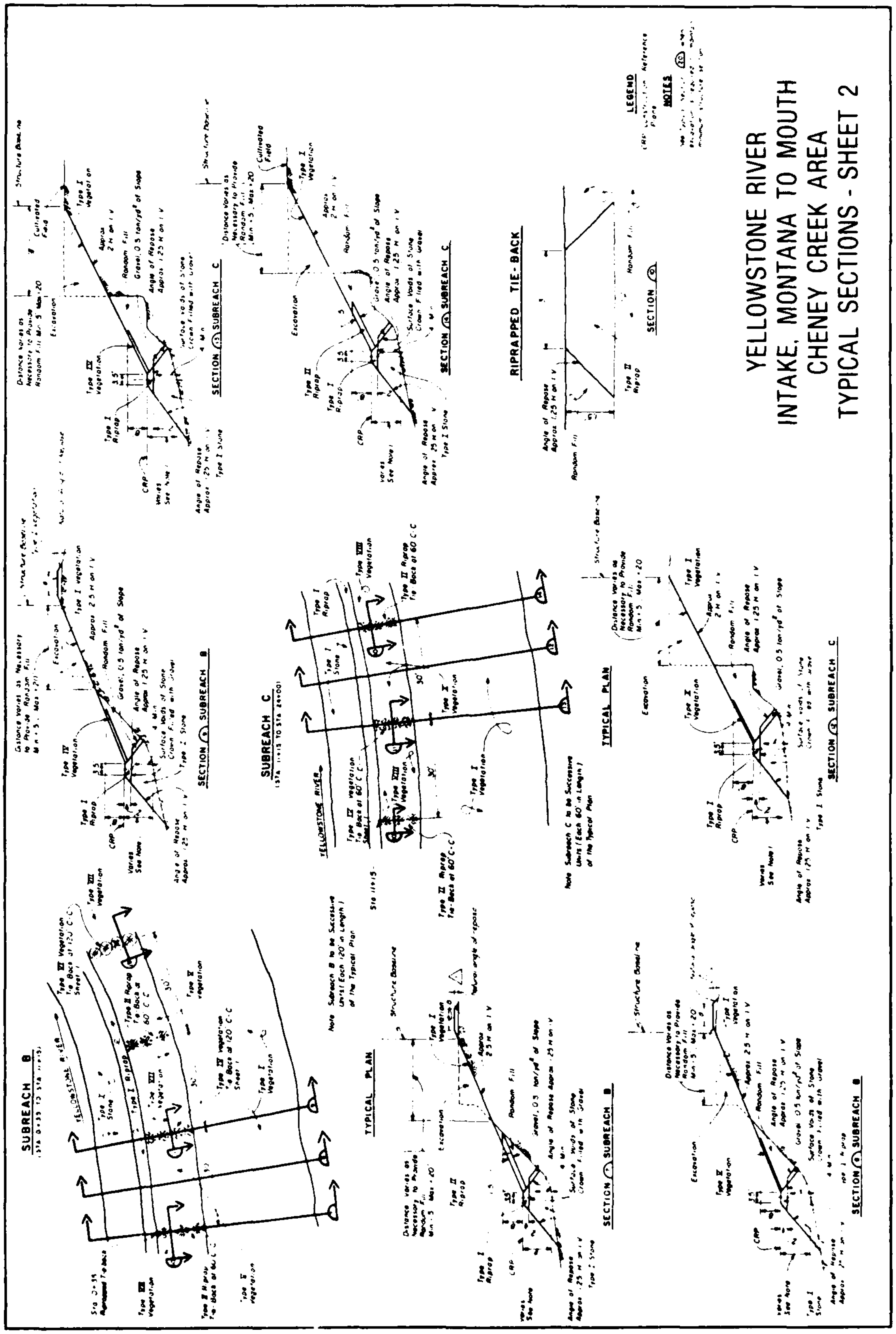
YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA
GENERAL PLAN AND
CROSS SECTION MEASUREMENTS

PLATE 21



YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA
TYPICAL SECTIONS - SHEET 1

PLATE 22



YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA
TYPICAL SECTIONS - SHEET 2
LEGEND
NOTES
SUBREACH B
SUBREACH C
TYPICAL PLAN
RIPRAPPED TIE-BACK

PLATE 23

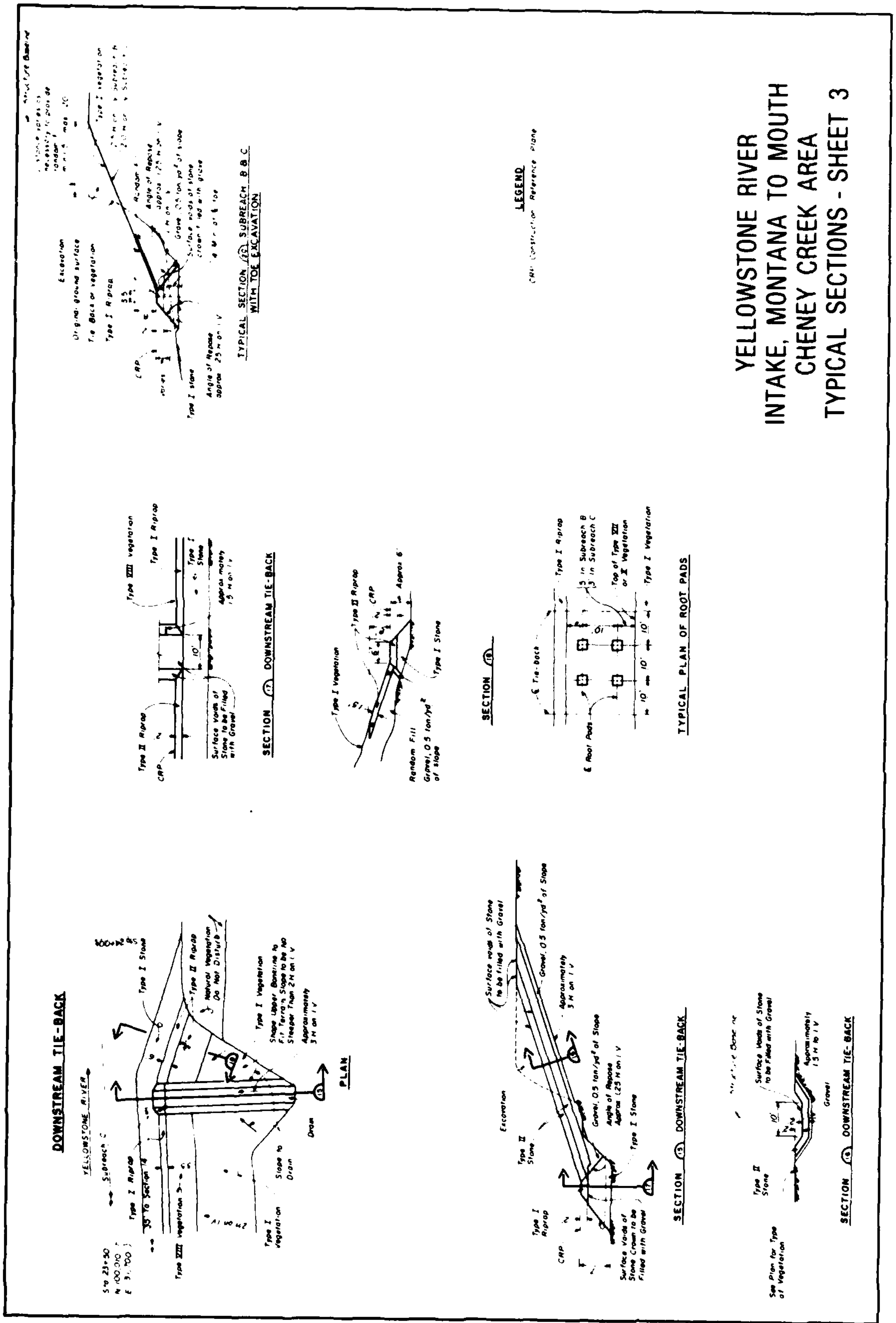

YELLOWSTONE RIVER
INTAKE, MONTANA TO MOUTH
CHENEY CREEK AREA
TYPICAL SECTIONS - SHEET 3
TYPICAL SECTION SUBREACH B & C WITH TOE EXCAVATION
LEGEND
SECTION DOWNSTREAM TIE-BACK
SECTION
TYPICAL PLAN OF ROOT PADS
DOWNSTREAM TIE-BACK
PLAN
SECTION DOWNSTREAM TIE-BACK
SECTION DOWNSTREAM TIE-BACK

PLATE 24

CONSTRUCTION PERIOD: September 1980 to November 1980*

*Construction 100% complete.

LEGEND
x = measurements scheduled
✓ = data obtained

YELLOWSTONE RIVER EROSION CONTROL DEMONSTRATION
MONITORING SCHEDULE - RIVER ROAD AREA
Richland County, Montana
12 December 1980

TASK	Executor of Task: *Corps of Engrs	Executor of Task: A/E	Executor of Task: Gen. Contr.	Executor of Task: Other Agency	Performance Period: Before Const.	Performance Period: During Const.	Performance Period: Comple-tion Const.	Post-Construction Frequency Period (Minimum): 1981	1982	1983 6/	1984 6/	1985 6/
A. MONITORING OF PHYSICAL FEATURES												
1. Bankline Location Survey	1,3		x		✓ x	✓ x	✓ x	x	x	x	x	x
2. Overbank/Streambank Cross Section	1,3		x		✓ x	✓ x	✓ x	x	x	x	x	x
3. ~~Velocity Measurements~~ 5/ NOT USED												
4. Structural and Vegetative Changes (Qualitative)	1,3,11					✓ x		x	x	x	x	x
5. Staff Gage Readings	1,2,11		x			x						
6. Hydrography 1/	1,2,3				✓ x		x		x		x	
7. ~~Surface Flow Patterns~~ NOT USED												
8. River/Erosion Conditions (Qualitative)	1,2,11				✓ x	✓ x	✓ x	x	x	x	x	x
B. MATERIALS TESTS												
1. Soil Analysis of Exposed Banks 5/	1,2,5,7				✓ x							
2. ~~Bed Material Analysis~~ 5/ NOT USED												
3. Construction Materials Durability (Qualitative)	1,2,7,11					x		x	x	x	x	x
4. Mechanical Analysis of Construction Materials 2/	1,2,7				x	x	x	AS DEEMED NECESSARY				
C. PHOTOGRAPHY												
1. Oblique	1,4		x		✓ x	x	x	x	x	x	x	x
2. Controlled Vertical	1,3		x		✓ x				x		x	
3. Ground Level 5/	1,4,11		x		✓ x	✓ x	✓ x	x	x	x	x	x

PLATE 25

LEGEND
x = measurements scheduled
/ = data obtained

YELLOWSTONE RIVER EROSION CONTROL DEMONSTRATION
MONITORING SCHEDULE - RIVER ROAD AREA
Richland County, Montana
12 December 1980

TASK	EXECUTOR OF TASK: *Corps of Engrs	A/E	Gen. Contr.	Other Agency	PERFORMANCE PERIOD: Before Const.	During Const.	Completion Const.	POST-CONSTRUCTION FREQUENCY PERIOD (Minimum): 1981 JFMAMJJASOND	1982 JFMAMJJASOND	1983[6/] JFMAMJJASOND	1984[6/] JFMAMJJASOND	1985[6/] JFMAMJJASOND
C. PHOTOGRAPHY (Cont'd.)												
4. Color Videotape (Aerial)	1,4							AS DESIRED NECESSARY				
D. BIOLOGICAL MONITORING												
1. Fish/Micro-Organism Sampling[2/]	6			x	/ x			AS DESIRED NECESSARY				
2. Terrestrial/Aquatic Habitat Changes (Qualitative)	6			x	/ x			x	x	x	x	x
E. REVIEWS												
1. Joint Field Inspections[3/]	1,11				/ x			x	x	x	x	x
2. Field Reports[4/]	1,6							x	x	x	x	x
3. Interim Reports	1,6							x x				x

FOOTNOTES

1/Includes Aggradation-Degradation, Bed Mapping, and other Hydrographic Data.
2/Micro-Organism Samples - Water, Natural Bank and Bed and Structure Surface
3/Other Agencies and Concerned Entities
4/Narrative Reports Prepared by Concerned Entities on Area Assessment of Present Conditions and Changes
5/This data to support engineering and biological determinations.
6/This work is dependent upon Congressional action to extend the nationwide Section 32 Program completion date and to authorize additional program funding.

***CORPS OF ENGINEERS**

1. Channel Stabilization Section
2. Water Quality & Sediment Section
3. Surveys and Mapping Section
4. Photo Unit
5. Foundations and Materials Branch
6. Environmental Analysis Branch
7. MRD Laboratory
8. Florence Area
9. Lewis and Clark Project Office
10. North Dakota Area
11. Montana Area

PLATE 26

CONSTRUCTION PERIOD: September 1980 to November 1980*

*Construction 100% complete.

LEGEND

x = measurements scheduled
✓ = data obtained

YELLOWSTONE RIVER EROSION CONTROL DEMONSTRATION
MONITORING SCHEDULE - CHENEY CREEK AREA
McKenzie County, North Dakota
5 December 1980

TASK	EXECUTOR OF TASK: *Corps of Engrs	A/E	Gen. Contr.	Other Agency	PERFORMANCE PERIOD: Before Const.	During Const.	Completion Const.	POST-CONSTRUCTION FREQUENCY PERIOD (Minimum): 1981	1982	1983[6/]	1984[6/]	1985[6/]
A. MONITORING OF PHYSICAL FEATURES												
1. Bankline Location Survey	1,3		x		✓ x	✓ x	✓ x	x	x	x	x	x
2. Overbank/Streambank Cross Section	1,3		x		✓ x	✓ x	✓ x	x	x	x	x	x
3. Velocity Measurements[3/]	1,2,3							x	x	x	x	x
4. Structural and Vegetation Changes (Qualitative)	1,3,11					✓ x		x	x	x	x	x
5. Staff Gage Readings	1,2,11		x			x						
6. Hydrography[1/]	1,2,3				✓ x		x		x		x	
7. ~~Surface Flow Patterns~~ NOT USED												
8. River/Erosion Conditions (Qualitative)	1,2,11				✓ x	✓ x	✓ x	x	x	x	x	x
B. MATERIALS TESTS												
1. Soil Analysis of Exposed Banks[5/]	1,2,5,7				✓ x							
2. Bed Material Analysis[5/]	1,2,5,7							x	x	x	x	x
3. Construction Materials Durability (Qualitative)	1,2,7,11					x		x	x	x	x	x
4. Mechanical Analysis of Construction Materials[2/]	1,2,7				x	x	x	AS DEEMED NECESSARY				
C. PHOTOGRAPHY												
1. Oblique	1,4		x		✓ x	x	x	x	x	x	x	x
2. Controlled Vertical	1,3				✓ x				x		x	
3. Ground Level[3/]	1,4,11		x		✓ x	✓ x	✓ x	x	x	x	x	x

PLATE 27

LEGEND
x = measurements scheduled
✓ = data obtained

YELLOWSTONE RIVER EROSION CONTROL DEMONSTRATION
MONITORING SCHEDULE - CHENEY CREEK AREA
McKenzie County, North Dakota
5 December 1980

TASK	EXECUTOR OF TASK				PERFORMANCE PERIOD			POST-CONSTRUCTION FREQUENCY PERIOD (Minimum)				
	*Corps of Engrs	A/E	Gen. Contr.	Other Agency	Before Const.	During Const.	Completion Const.	1981 JFMAMJJASOND	1982 JFMAMJJASOND	1983 6/ JFMAMJJASOND	1984 6/ JFMAMJJASOND	1985 6/ JFMAMJJASOND
C. PHOTOGRAPHY (Cont'd.)												
4. Color Videotape (Aerial)	1,4							← AS DEEMED NECESSARY →				
D. BIOLOGICAL MONITORING												
1. Fish/Micro-Organism Sampling 2/	6			x	✓ x			← AS DEEMED NECESSARY →				
2. Terrestrial/Aquatic Habitat Changes (Qualitative)	6			x	✓ x			x	x	x	x	x
E. REVIEWS												
1. Joint Field Inspections 3/	1,11				✓ x			x	x	x	x	x
2. Field Reports 4/	1,6			x				x	x	x	x	x
3. Interim Reports	1,6							x				x

FOOTNOTES

1/Includes Aggradation-Degradation, Bed Mapping, and other Hydrographic Data.
2/Micro-Organism Samples - Water, Natural Bank and Bed and Structure Surface
3/Other Agencies and Concerned Entities
4/Narrative Reports Prepared by Concerned Entities on Area Assessment of Present Conditions and Changes
5/This data to support engineering and biological determinations.
6/This work is dependent upon Congressional action to extend the nationwide Section 32 Program completion date and to authorize additional program funding.

***CORPS OF ENGINEERS**

1. Channel Stabilization Section
2. Water Quality & Sediment Section
3. Surveys and Mapping Section
4. Photo Unit
5. Foundations and Materials Branch
6. Environmental Analysis Branch
7. MRD Laboratory
8. Florence Area
9. Lewis and Clark Project Office
10. North Dakota Area
11. Montana Area

PLATE 28

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
Billings Area Office
Federal Building, Room 3035
316 North 26th Street
Billings, Montana 59101

IN REPLY REFER TO:
ES

January 22, 1981

U.S. Army District, Omaha
Corps of Engineers
6012 U.S. Post Office and Courthouse
Omaha, NE 68101

Dear Sir:

This letter is our interim Fish and Wildlife Coordination Act Report concerning work on the lower Yellowstone River in Montana and North Dakota under Section 32 of the Water Resources Development Act of 1974. Originally, three demonstration project sites were selected - "River Road" (river mile 28.9), "Cheney Creek" (river mile 20.4), and "Horse Creek" (river mile 12.3). Construction has not occurred on the Horse Creek site, and according to our understanding, may not for some time; therefore, only the River Road and Cheney Creek sites will be discussed in this report.

Previous understanding between the Corps and FWS included an agreement that up-to-date aerial photographs would be supplied to this office to assist in our assessment of local project impacts. The photographs were not received in time for use in this interim report, which is due to your office February 1, 1981. The photographs, received January 19, will be used in the preparation of any additional reports on the project sites as may be appropriate.

The lower Yellowstone River is an everchanging, meandering stream, in many respects typical of the rivers found in the northern prairie region. Freezing and thawing, shifting and movement of ice, and high spring flooding influence the erodible soils of the river valley. These hydraulic forces frequently move the river channels, create islands, and produce side channels. The complex riverine system that results is dependent upon these forces and provides important habitats for numerous aquatic and riparian fauna. White-tailed deer, raccoon, skunk, beaver, mink, weasel, porcupine, rodents, waterfowl, upland game, raptors, and song birds are typically found in this habitat. Bald eagles and whooping cranes, two Federally-listed endangered species, also periodically frequent this riparian area. Sauger, walleye, burbot, crappie, channel catfish, black bullhead, paddlefish, drum, sturgeon, northern pike, carp, goldeye, and several dace, minnows, shiners, and chubs are found in the lower reach of the Yellowstone.

EXHIBIT I (SHEET 1 OF 4)

Since the onset of development along the Yellowstone River, various methods have been pursued by man to control the erosive nature of the river. These methods have been generally ineffective, are expensive, and many of them are rather unpleasant aesthetically. The Section 32 work reported on herein is reportedly an attempt to develop relatively inexpensive bank protection measures that would present a more natural appearance, once in place.

River Road Site

This project consists of constructing a 500 linear-foot jetty across a side channel. The jetty has recently been completed and extends from the south side of the river to a sand and gravel island. In recent years, water flowing down this channel has eroded the bank, threatening the adjacent county road, a buried pipeline, and an old concrete pump house. The jetty is constructed of a gravel core which is buried under sand. A gravel blanket covering the entire structure is designed to protect against water and ice erosion. Grass seed and bulrush sprigs have been planted to help hold the dike together. Approximately 700 cubic yards of clean gravel from a local quarry and 6,150 cubic yards of sand from the adjoining island were used to construct the jetty.

Although the vegetation has not yet had a chance to grow and develop, the jetty is fairly pleasing in appearance. The lack of large rock or concrete revetment allows the jetty to blend in with the local environment. Minimal impact to the aquatic ecosystem is expected in that construction took place during the low water season, equipment was not permitted in the wetted channel, and clean material was used for fill. Some adverse impacts to local fall spawning fish populations may have occurred due to actual disturbance of the substrate and downstream turbidity affecting developing larval fish. These local impacts, however, were probably of little significance.

Construction of the access road and turnouts at the project site required the removal of some riparian vegetation. However, according to project design, these disturbed areas are to be seeded; therefore, they should only be temporarily devegetated, especially if the access road is closed to all use. Also, willows should naturally re-establish in these areas in a short period of time.

Comparing the local section of the river at present with the situation in 1979, it appears that the main channel is moving north, the islands adjacent to the jetty are becoming larger, and the channel of concern is decreasing in size. In other words, it seems evident that the erosion problem next to the county road would eventually disappear even without the new structure, as the channel is naturally aggrading and was progressively conveying less water prior to jetty construction.

EXHIBIT I (SHEET 2 OF 4)

To minimize costs and to aid in blending with the surrounding environment, the jetty was constructed without large riprapping or other "state of the art" protection against erosion. Hence, the structure may not hold up against winter ice movement and peak spring run-off. Consequently, we are somewhat concerned about the possible need to continually repair the jetty, thus keeping the area in a constant state of disturbance.

Cheney Creek Site

The design for this site involves the construction of approximately 2,400 linear-feet of reinforced composite revetment with tiebacks at several strategic locations. Willow sprigs and grass seed were planted to help stabilize the surface materials. This project is designed to protect the streambank, adjoining irrigated field, and the existing Cheney Creek Road. Reportedly, the rate of erosion has averaged about 21 feet per year over the last 78 years, with some sections recently experiencing as much as 50 feet or more of erosion in a year.

Shaping the bank and planting of vegetation was designed to help maximize bank protection and to blend in the construction with the surrounding environment. The project is, in fact, quite pleasing to the eye and will be more so as the vegetation grows and develops.

Impacts to the aquatic environment were limited to the physical reshaping of the bank and placement of the rock toe. Turbidity and inundation of substrate by rock may have contributed to minor impacts, but these were short-lived. In the long run, the local area will benefit as turbidity caused by continual sloughing of the bank during high water and during the irrigation season should cease.

There was no woodland or wetland habitat disturbed by construction. A wooded area on the downriver end of the project was not included in the project and, therefore, was virtually left alone. Disturbance to the irrigated field and adjoining grasslands may have displaced associated wildlife, but this should be only temporary.

The ability of this project to withstand the erosive forces of ice movement and high water are not known and will become evident only after several hydrologic seasons. We are concerned about the possibility of continual maintenance being needed, thus keeping the area frequently disturbed.

Impacts to fish and wildlife habitat from both of these stream bank protection projects appear very minor and insignificant at a local level. The possible influence these and similar projects may have on river reaches downstream, by altering the hydraulics of the river, does raise some concern. For example, serious erosion is becoming

EXHIBIT I (SHEET 3 OF 4)

evident across the river and downstream from the River Road site, with large cottonwood trees (important to riparian wildlife) being lost at an increasing rate. The installation of the jetty may accellerate this rate of tree loss by diverting more water to the north side of the river channel.

We are, however, much more concerned about the precedent that may become established along the lower Yellowstone River. If a stretch of river bank must be protected, we would prefer use of the methods used in these demonstration projects over historic methods, based on the results thus far. However, attempting to physically control bank erosion on a large, highly dynamic river usually just shifts the problem to adjacent locations. Combining this fact with the possibility of having less expensive means available for bank protection could eventually incourage a great many bank "control" efforts, which could progressively confine and narrow the river channel, to a point where the quality and amount of aquatic and riparian environments of the lower Yellowstone River would be seriously reduced. Increased intrusion of man-made developments on the flood plain would also be encouraged.

We appreciate the opportunity to comment.

Sincerely,

Wally Steucke

Wally Steucke
Area Manager

cc: Director, Montana Department of Fish, Wildlife, and Parks, Helena, MT (5)
Al Elser, Montana Department of Fish, Wildlife, and Parks, Miles City, MT (2)
Supervisor, ES, USFWS, Bismarck, ND (2)
Regional Director, USFWS, Denver, CO (ENV) (2)
Director, USFWS, Washington D.C. (ENV) (2)

EXHIBIT I (SHEET 4 OF 4)

EEL RIVER NEAR FERNBRIDGE, CALIFORNIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

EEL RIVER AT SINGLEY POOL NEAR FERNBRIDGE, CALIFORNIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location: The Eel River Demonstration Project for Streambank Erosion Control is located on the right bank of the Eel River, approximately one mile west of Fernbridge, California. The location of the site is shown on Plate 1.

2. Authority: Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251, as amended by Section 155 of the Water Resources Development Act of 1976.

3. Purpose and Scope: This report describes the bank erosion problem and the types of bank protection used, and evaluates the performance of the demonstration project, which was constructed and monitored by the San Francisco District.

4. Problem Resumé: The demonstration site is located on the right bank of the Eel River near the head of tidal action. Serious erosion has occurred along the site. The bank has eroded about 600 feet since 1968. Pictures of the eroding bank at the site are shown on Plate 2. The banks at the site extend about 15 feet above the low-flow water surface. The upper 5 feet of the banks are nearly vertical, and the slope of the lower 10 feet varies from 1V to 1H to about 1V on 2H. The streambed is about 5 feet below the low-flow

water surface. The soil at the site is mostly silt with some sand and clay that has very little resistance to erosive forces. During the site selection phase, it was observed in the field that a major cause for streambank erosion at this site has been the continual loss of the toe of the slope when subjected to floodflows.

II. HISTORICAL DESCRIPTION

5. Stream:

a. Topography. The Eel River Basin contains a series of rugged, deeply trenched, northwest-trending ridges and mountains that are controlled by folding and faulting. The mountains along the east side of the basin rise to elevations of from 5,000 to 7,500 feet NGVD* and those in the western part of the basin range from 1,000 to 2,000 feet in altitude. Flood plains begin to appear in the lower reaches of major tributaries and progressively widen on the main stem. Near the mouth, the Eel River flows through a wide, flat delta which has elevations ranging from 0 to 20 feet NGVD.

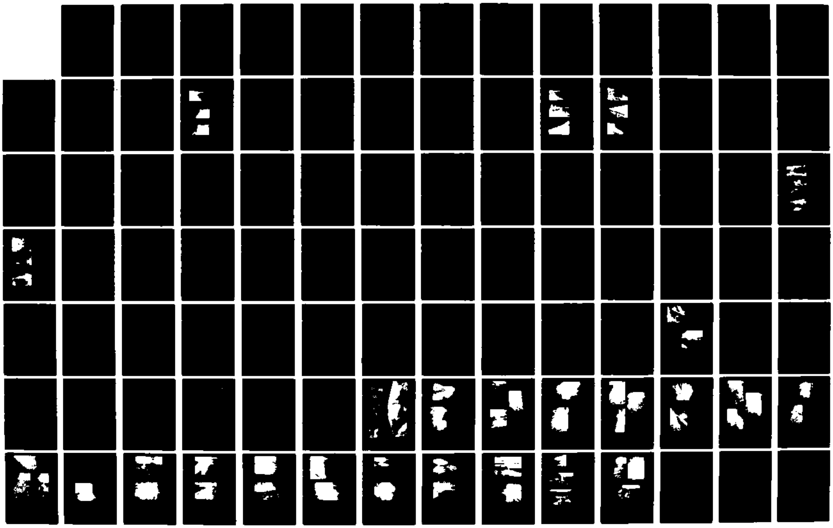
b. Geology. Over 90 percent of the Eel River basin area is underlain by Late Jurassic to Late Cretaceous rocks of the Franciscan assemblage. Graywacke sandstones containing subordinate amounts of shale and conglomerate are the principal rock types in the assemblage, but interlayed submarine volcanic rocks of extrusive origin and serpentine and related rocks are important constituents. All of the Franciscan rocks have been subjected to several episodes of intense folding and faulting during the geologic past. A thick accumulation of Tertiary and Quaternary marine sedimentary rocks overlies the Franciscan assemblage in the delta area at the downstream end of the basin. Subsequent compressional forces in the earth's crust folded

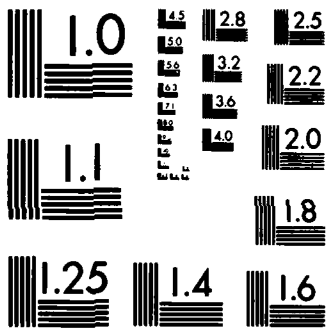
---

* All elevations in this report refer to National Geodetic Vertical Datum unless otherwise noted.

AD-A121 138 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. 2/4

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

these sediments into a broad, east-west-trending syncline which was later filled with deltaic deposits.

c. Hydrologic Characteristics. Floodflows on the Eel River rise very rapidly. Peak discharges at Fernbridge from a flood-producing storm occur about 18 hours after the highest intensity rainfall of the storm. Inundation begins in the delta area in the proximity of Fernbridge at a discharge of about 150,000 cubic feet per second (c.f.s.). Duration of the flood flows may last from four to five days. Normal annual precipitation within the basin ranges from about 40 inches in the Eel River delta to 100 inches near the crest of the westerly ridge of the South Fork Eel River. The average annual precipitation for the entire basin is about 57 inches.

The peak discharge vs frequency relationship for the Eel River in the Fernbridge area, as developed from peak discharge data collected at the U.S. Geological Survey stream-gaging station, Eel River at Scotia, is as follows:

Event	Eel River at Fernbridge, c.f.s.
100-year	750,000
50-year	640,000
10-year	400,000
5-year	300,000
2-year	170,000

The Standard Project Flood (S.P.F.) on the Eel River is estimated to be 920,000 c.f.s. at Fernbridge.

d. Streams and Stream Characteristics. The principal tributaries of the Eel River are the Van Duzen River, North Fork Eel River, Middle Fork Eel River and South Fork Eel River, which drain a

total of about 60% of the basin. The Eel River has a streambed slope ranging from about five feet per mile near the Van Duzen River to about 22 feet per mile at the Van Arsdale Dam, which is at river mile 150. Slopes of Eel River tributaries are generally steeper than those of the main stem. The basin drainage pattern is principally trellis, and although the major streams are chiefly parallel to the structural grain of the area, in some places they are markedly transverse. The Eel River and its tributaries flow through narrow, steep-walled, V-shaped canyons throughout most of their lengths. Their channel sections vary widely, however, because of the canyon type topography of the upper watershed. In the delta area, near the mouth of the Eel River, the channel is about 20 feet deep and about 1,000 to 2,000 feet wide above Fernbridge. Downstream from Fernbridge, the river flows into an estuary. The channel capacity of the river at Fernbridge is about 150,000 c.f.s.

e. Environmental Considerations. The environmental aspects of the project, including water quality, vegetation, fish and wildlife, benthos, rare and endangered species, and cultural resources, were carefully reviewed and coordinated prior to the construction of the project. During the site selection and design process, serious consideration was given to minimize the disturbance of native vegetation and wildlife habitat. This aspect was effected by joint field reconnaissance with environmental representatives of Humboldt County as well as personnel from the State Department of Fish and Game and the U.S. Fish and Wildlife Service. The proposed project was reviewed by the above agencies as well as by the North Coast Region of the California Regional Water Quality Control Board and the North Coast Regional Commission of the California Coastal Commission.

An Environmental Assessment was prepared in accordance with Section 404(b) of Public Law 92-500. Further, a public notice which described the proposed project was prepared and distributed to about

150 Federal, State and local agencies and officials, property owners in the vicinity of the project, and other interested individuals.

In addition, in accordance with the requirements of the National Environmental Policy Act of 1969 (Public Law 91-190), the Corps of Engineers evaluated the environmental aspects of the proposed activity. From an analysis of these impacts, it was determined that the activity would have no significant effect on the quality of the environment. Therefore, an Environmental Impact Statement was not prepared.

6. Demonstration Site - Test Reach:

a. Hydraulics and Hydrology. The slope of the Eel River at the demonstration project site is about four feet per mile. Heavy rainfall and accompanying runoff from the 3,600- square mile upstream drainage basin is seasonal. Most of the heavy rainfall and runoff occur during the months of November through March. Flows in the river have ranged from less than 100 c.f.s. in the dry season to about 800,000 c.f.s., which was in excess of a 100-year event, in December 1964. During the 1964 flood, water flowed about 10 to 12 feet deep over the top of the bank. It is estimated that the maximum velocities along the front of the works will approach 14 feet per second (f.p.s.) and that the velocities may reach seven f.p.s. through and behind the works during extreme events such as the December 1964 flood.

b. Riverbank Description.

1. Bank Materials. The bank height along the demonstration site is about 20 feet consisting primarily of alluvial terraces. The upper four to five feet is near vertical and consists primarily of medium stiff silt. The lower bank slopes at about 1V on 2H. The soils in the lower part of the bank consist of loose,

silty sandy gravel, silty gravelly sand, and gravelly sand with lenses of soft to medium stiff sandy silt and sandy clay. Soils below the riverbed consist of loose to dense silty, sandy gravel and occasionally cobbles with intermittent lenses of soft to stiff sandy to gravelly clay. The maximum size of cobbles is about four inches.

2. Normal Bank Vegetation. Vegetative cover on the banks of the demonstration site are grasses and occasional willows. Due to the highly erosive quality of the existing bank material, vegetative growth has been very sparse.

3. Bank Erosion Tendencies. The banks along the test site have been eroding at variable rates. The severity of the erosion process increases with the stage attained during flood flows. Compounding the erosion process is the inability of the toe of bank slope to retain the upper bank during flood flows. Pictures of the eroding banks of the site are presented on Plate 2. The test site is situated on the outside of a curve with a radius of about one mile; the channel width is about one-half mile. Within bank flows meander between the banks of the river and attack the outside of bends and build bars along the inside of bends. The river in recent years has meandered over a width of about one-half mile in the proximity of the test site. Erosion problems exist at many locations in the delta area, which extends from the mouth of the river to above the confluence with the Van Duzen River. During the site selection phase, it was observed in the field that a major cause for streambank erosion at this site has been the continual loss of the toe of the slope when subjected to high flows.

III. DESIGN & CONSTRUCTION

7. General:

This project was formulated on the theme of testing innovative, environmentally acceptable methods of controlling streambank erosion

with locally readily available materials in lieu of the conventional riprapping of sideslopes. Methods were developed which would be compatible with the type of streamflow and streambank conditions at the demonstration site. The Eel River drains a watershed predominantly covered with timber and as such introduces a considerable quantity of floating debris during significant runoffs. Consequently, it was concluded that this demonstration site should encompass a means of protecting the streambank from floating debris. Within this framework, four methods adopted for this site were developed and constructed. The methods selected were: Rock Hardpoints, two variations of Timber Pile Fence, and Rock Toe Protection. As a concurrent benefit from the pile fencing method of protection, it was anticipated that deposition of silt and debris would occur.

8. Basis for Design: The design was based upon engineering judgment, observed effectiveness of comparable existing bank protection measures, inspection of the site, topographic surveys, soils and foundation investigations, and an estimate of forces that could reasonably be imposed upon the component parts of the structures. Improvements were designed to provide protection during a 10-year event (400,000 c.f.s.) in the river. The location of each of the bank protection works is shown on Plate 3. The following paragraphs present design details for the different types of bank protection works which were constructed.

a. Rock Hardpoints. Rock hardpoints were designed to resist anticipated flow velocities. Groins in the shape of a trapezoid were placed in five-foot deep trenches which extended below streambed, as shown on Plate 4. Plate 9 shows the rock hardpoints during flow conditions. Rock gradation for the hardpoints was based on experience gained by other participants in the Section 32 Program. Maximum velocities associated with the design discharge were expected to range between 11 and 12 f.p.s. adjacent to the bank being protected.

b. Pile Fence. The pile fence was designed to resist forces produced by the bouyancy of the wood piles and wood bracing, the river currents, and the impact of floating debris. It was estimated that the maximum velocity along the river face of the structure associated with the design discharge would be 12 f.p.s. and that the velocity through the structure would be 7 f.p.s. To prevent possible undermining of the pile fence structure, quarry-run rock was placed as shown on Plate 5 for the full length of this mode of streambank protection. The wire fence along the channel face of the structure, which would be supported by the piling, would tend to collect debris to reduce and deflect river currents along the fence. The chance of direct impact by floating debris would also be reduced by the wire fence. This factor, as well as the fact that currents would run parallel to the structure, led to the decision to design the piles and wood bracing for glancing blows, not direct impact by large floating debris in the river. It was assumed that the maximum impact force from floating debris would be equivalent to the force produced by a floating log, two feet in diameter and 25-feet long, moving at five f.p.s., impacting the structure. The Douglas fir wood bracing was designed for a compression parallel to grain strength of 1,250 pounds per second inch (psi) and a bending strength of 1,500 psi. The wood piles were designed for compressive and bending stresses of 2,450 psi. The allowable skin friction force for the wood piles was determined to be about 1,500 psi from the soils and foundation investigations. The timber pile fence protection was installed in two configurations, light and dense. Light and dense pile configurations were installed at 12 feet and 9 feet centers, respectively. Additionally, the dense pile fence received heavier bracing between piles. The objective of using the two configurations was to determine the structural adequacy and to observe the effectiveness to trap debris and sediment of each configuration. Two types of four-foot high wire fence were used with the fence: heavy galvanized field fencing with widely spaced wires was used with the dense pile fence, and six-inch square W4 x

W4 welded wire mesh was used with the light pile fence. The details of each type of fencing are presented on Plates 5 and 6. The pile fence as constructed at the site is shown on Plate 8.

c. Toe Protection. Toe protection was designed to prevent the toe of the existing streambank from being eroded during more frequently occuring floodflows. Sizing of the riprap was based on anticipated localized velocities with analyses similar to that used for the rock hardpoints. The streambank was not protected by conventional methods above elevation 8 N.G.V.D. It was anticipated that the bank could be established by the toe protection below elevation 8 in conjunction with the planting of willows and voluntary growth of vegetation above elevation 8. This approach was furthered with the notion that streamflow velocities in the upper regions of the streambank would be lower than at the toe. Plate 7 presents the details of this type of protection. Plate 7 presents the details of this type of protection. The rock toe protection as constructed is shown on Plate 8.

9. Construction Details: Four different types of bank protection were constructed at the site, as shown on Plate 3. The total project length was 2,500 feet. The upstream 800 feet consisted of rock hardpoints; the middle 800 feet consisted of 400 feet of light pile fence and 400 feet of dense pile fence. The downstream 900 feet consisted of rock toe protection. A general description of each type of streambank protection used is contained in the following paragraphs.

a. Rock Hardpoints. Rock hardpoints were placed along the upstream 800 feet of the test reach. Details are shown on Plate 4. The rock hardpoints were formed by the placement of quarry-run rock in a trapezoidal excavation having a five-foot bottom width, a depth of five feet, and side slopes of 1V on 1.5H. Quarry-run rock extended from the top of the bank to -7 feet N.G.V.D. The rock

hardpoints were placed on 40-foot centers, and extended from the toe of the existing bank to the top of the bank at a slope of 1H on 2V. Quarry-run rock possessed qualities such that the largest stone did not weigh in excess of 400 pounds and 50% of all individual stones weighed over 150 pounds.

b. Pile Fencing.

1. Light Pile Fence. The light pile fence consisted of 26-foot long timber piles driven at least 12 feet into the ground. Maximum penetration of timber piles was limited to about 16 feet. The piles, which were Douglas fir and which had been debarked and pressure treated, had a minimum tip diameter of eight inches and a minimum butt diameter of 12 inches or greater. The pile fence consisted of piles at 12-foot centers. Each pile was tied to adjacent piles, as shown on Plate 5, by pressure treated timber members. The timber members varied from two inches by 10 inches to three inches by 12 inches in size. The timber members transmitted impact loads to a second row of piles located about six feet behind (landward) of the 26-foot long piles. he second row of piles consisted of piles about 16 feet in length c hich at least eight feet were driven into the ground. Each of the piles in the second row were placed downstream of the piles in the front row. Each set of piles was tied together by one horizontal and one diagonal timber member. This configuration of piles and timber members enabled the structure to resist impact forces in a more capable manner. The timber members were connected to the piles by galvanized 3/4-inch steel bolts, washers and nuts. To collect debris, to reduce velocities, and to encourage sediments to be deposited behind the piles, a four-foot high wire mesh fence was attached to the bottom of the outer row of piles. An apron of quarry-run rock possessing qualities such that the largest stone did not weigh in excess of 200 pounds and 50% of all individual stones weighed over 50 pounds was placed in front of the piles. Rock thickness was 24 inches above mean higher high

water and 36 inches below mean higher high water. The elevation of the rock toe was set at -7.0 feet N.G.V.D.

2. <u>Dense Pile Fence</u>. The dense pile fence, as shown on Plate 6, was a modification of the light pile fence. Piles were driven at nine-foot centers. However, they were tied to the back row of piles in both the downstream and upstream directons. It was believed that this would create a stronger structure and encourage more sediments to be deposited behind the pile fence by the additional resistance to flow created by the triangular shaped bracing pattern. To collect debris, to reduce velocities and encourage sediments to be deposited behind the piles, a four-foot high wire mesh fence was attached to the bottom of the outer row of piles. The upstream end of dense pile fence was tied back to the bank. An apron of quarry-run rock was also placed in front of the dense pile fence.

3. <u>Rock Toe</u>. The rock toe protection was placed immediately downstream of the pile fencing. This method of erosion control, as shown on Plate 7, employed a continuous mat of riprap between elevations +8 and -7 N.G.V.D. The riprap was placed on a 1V on 2.5H side slope with a minimum horizontal bottom width of five feet. Riprap thickness was 18 inches between elevation +8 feet, N.G.V.D., and mean higher high water, and 27 inches between mean higher high water and -7 feet, N.G.V.D. Riprap gradation was as follows:

Stone Weight (pounds)	Percent Lighter by Weight
160	100
100	80 - 100
50	45 - 80
20	15 - 45
5	0 - 15

10. Cost. The total cost of construction and all other associated costs to date for the four methods of streambank protection was $487,200. It is estimated that engineering and design costs, and supervision and administration costs (E&D and S&A), which are included in the total project cost, were $73,000 and $43,000, respectively. The total costs associated with each type of streambank protection employed at the demonstration site to date are as follows:

Method of Protection	Cost Per Foot (Including E&D plus S&A)	Cost Per Foot (Contract Cost Only)
Rock Toe Protection	$148.00	$116.00
Rock Hardpoints	$138.00	$102.00
Light Timber Pile Fence	$268.00	$196.00
Dense Timber Pile Fence	$341.00	$269.00

At no cost to the Government, the planting of willow cuttings on the streambank was scheduled at the completion of the construction contract under the joint efforts of the State Department of Fish and Game and the California Conservation Corps (C.C.C.). Due to the early onset of high flows in the fall of 1979, planting did not take place until the fall of 1980. A site inspection in April 1981 indicated very limited growth of the plantings. Willows and alders were, therefore, replanted in May 1981. These plantings were concentrated in the reach which used the rock toe method of streambank protection.

11. Monitoring Program. The monitoring program was developed in accordance with guidelines provided in the Waterways Experiment Station report "Guidelines for Monitoring and Reporting Demonstration Projects" dated September 1977 and in accordance with discussions with the South Pacific Division office representative on the Section 32 Steering Committee. The goal of the monitoring program was to observe and document the performance of the methods

employed and to determine the reasons for any failure. The monitoring program is summarized in Plate 11. The site has been monitored for twenty months, including two flood seasons. A chronological listing of significant events occurring at the site since construction follows:

a. Date: 9/79 - Project construction completed.

b. Date: 10/79 - The first substantial flow in the Eel River for the season; estimated flow rate of 40,000 c.f.s. No damage was substained by the demonstration project.

c. Date: 2/80 - Inspection of the site after recession of flood flow of 12-14 January 1980. The estimated peak flow experienced was 250,000 c.f.s. which significantly exceeded the bankfull capacity of about 150,000 c.f.s. A single row of timber piles which formed a tie-back to the streambank at the upstream end of the timber pile fence was lost. Erosion of the streambank downstream of the rock toe protected reach due to eddy currents was observed. Bank loss along the timber pile and rock toe protected reaches was noted. Bank loss along the rock hardpoint reach was minimal.

d. Date: 5/80 - Low flow inspection. A noticeable loss of streambank was noted

caused by the sloughing of saturated material during drawdown of river stage. Minor damage to timber members and connections on pile fence was observed.

e. Date: 9/80 - Willow planting accomplished by joint efforts of the California Department of Fish and Game, California Conservation Corps, and the Corps of Engineers.

f. Date: 11/80 - Remedial repairs to timber pile and rock toe protected reaches accomplished.

Stream velocities along the demonstration site were measured in the field and ranged between 5 to 6 feet per second for flows at about 40,000 to 60,000 c.f.s. Velocity checks at the peak flow of 250,000 c.f.s. were not made.

12. Evaluation of Protection Performance. Since completion of the construction phase, the demonstration project has experienced flows ranging from 30,000 to 250,000 c.f.s. The largest discharge of 250,000 c.f.s. was experienced 12-14 January 1980. All experienced flow rates with the exception of the largest discharge flowed at depths less than one-half of the total bank height and streambank damage was minor. The 250,000 c.f.s. flow completely inundated the test site and overtopped the bank along the rock hardpoint reach. Bank loss was noted along the timber pile fence and the rock toe protection reaches. The most severe bank erosion occurred along the upstream end of the pile fence reach where approximately 30 feet of streambank was lost. The piles forming a tie-back to the bank for the timber pile fence method were lost during the highest flow.

Minor erosion has caused scalloping between rock hardpoints, which was anticipated. The amount of streambank loss due to the scalloping effect is localized to the lower one-third of the streambank. The lateral displacement of the rock forming the nearby rock hardpoint is expected to reinforce the weakened area. The condition of the demonstration project following the peak flow of 250,000 c.f.s. on 12-14 Jan. 1980 is shown on Plate 9.

A condition survey of the entire test reach was performed in June 1980 to document the damage which had occurred the previous winter. A comparison of pre-project and post project conditions at selected cross-sections along the bank are shown on Plate 10. Rehabilitation of the project is described in the following paragraph.

13. Rehabilitation: A contract to repair damage incurred during the previous winter season on the test reach was awarded on 17 October 1980. The work consisted of the following:

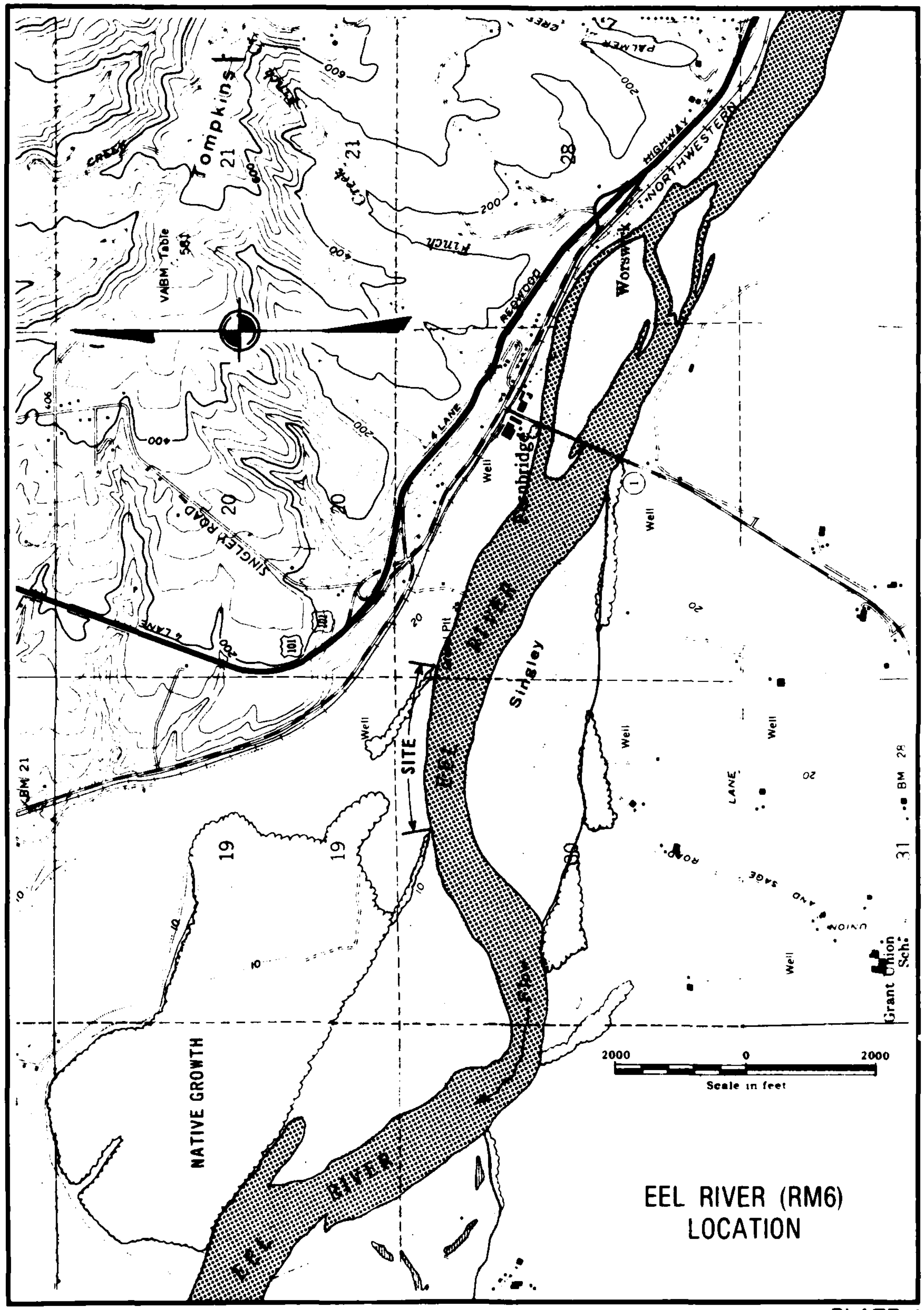
a. Constructing a timber pile groin where the timber pile fence and rock hardpoint sections meet. This groin replaced the timber pile tie-back which was originally constructed but which failed during the high flows of January 1980. The objective was to reduce velocities and encourage collection of debris.

b. Extending the downstream end of the rock toe reach with a rock tie-back to the existing streambank. The original terminus of the rock toe reach was subjected to streamflow conditions which formed erosive eddy currents and caused streambank loss. This erosive activity resulted in an unsatisfactory condition at the end of the test reach. The repair was made to inhibit further loss of existing streambank.

c. Total contracted cost for the repair work was $28,800.

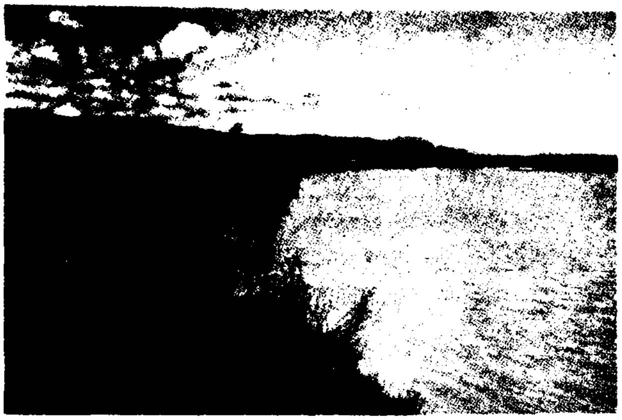
14. Conclusion: The overall performance of the demonstration project to date is considered to be successful. No damage has occurred along the toe of the streambank of the entire test reach. Some loss of upper streambank materials along the timber pile fence and rock toe reaches has occurred. The process of saturation sloughing and subsequent washing away of the toe material predominates at this site. Locations of severe erosion are the result of localized turbulence and high velocities. This loss is primarily attributable to the lack of vegetation to impede the erosive forces of the river. It is anticipated that vegetation, once it becomes established, will greatly reduce erosion.

A dense planting of willow cuttings was accomplished in early September of 1980. Due to poor growth, the area was replanted in May 1981. The planting was done on all unprotected, natural surfaces of the streambank along the entire test reach. It is hoped that the willow cuttings will proliferate and encourage silt deposition. The growth of willows is expected to enhance deposition of silt and control the loss of the streambank material.



EEL RIVER (RM6)
LOCATION
Scale in feet
2000
0
2000
SITE
EEL RIVER
NATIVE GROWTH
Singley
Fernbridge
Worswick
Tompkins
Finch
REDWOOD
HIGHWAY
NORTHWESTERN
4 LANE
SINGLEY ROAD
VABM Table
581
Grant Union Sch.
Well
Pit
BM 21
BM 28

PLATE 1



HARDPOINT REACH

TIMBER PILE REACH

ROCK TOE REACH

EEL RIVER (RM6)
PRE-PROJECT CONDITIONS

PLATE 2

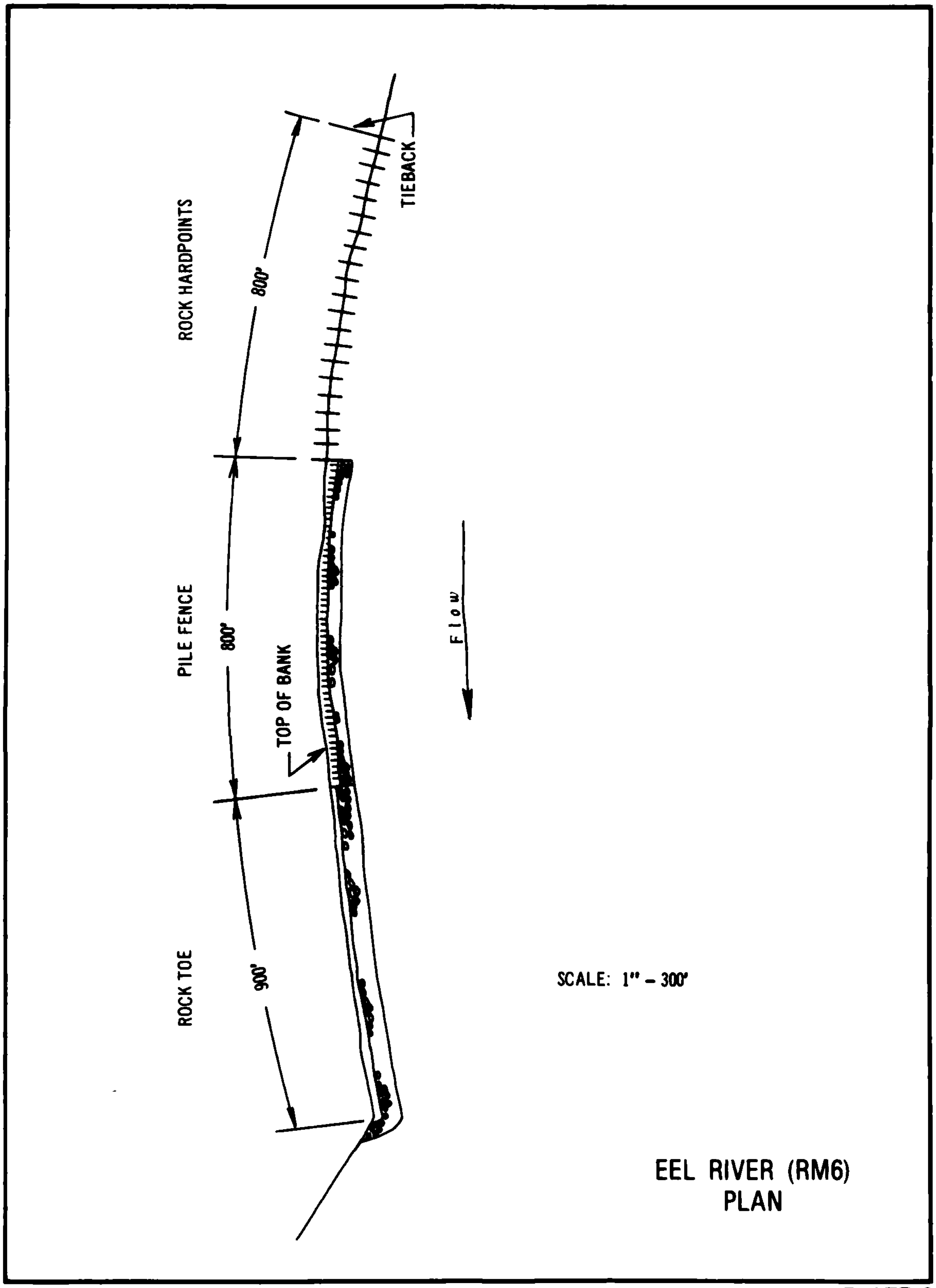
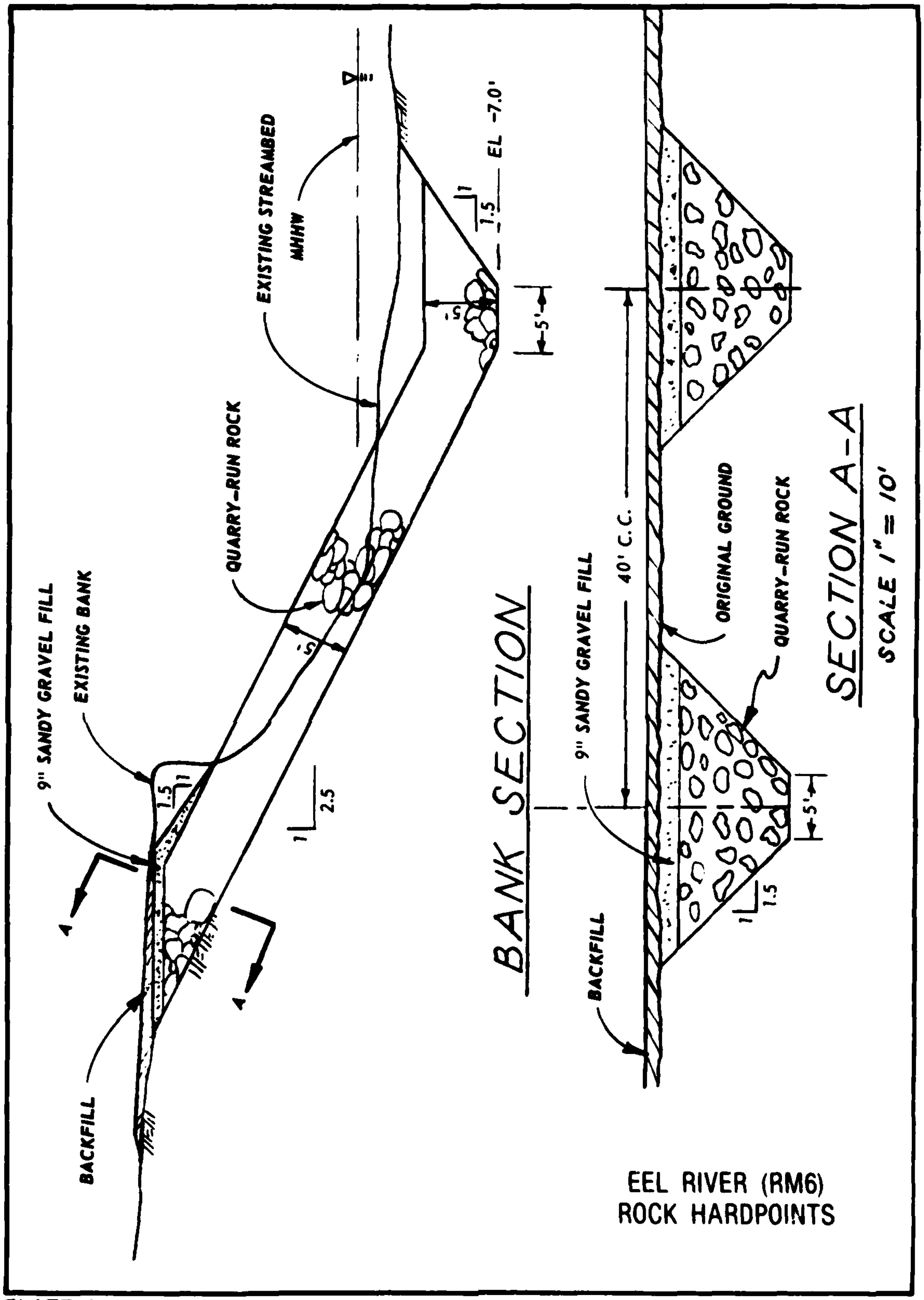

ROCK HARDPOINTS
800'
TIEBACK
PILE FENCE
800'
TOP OF BANK
Flow
ROCK TOE
900'
SCALE: 1" - 300'
EEL RIVER (RM6)
PLAN

PLATE 3



EEL RIVER (RM6)
ROCK HARDPOINTS

PLATE 4

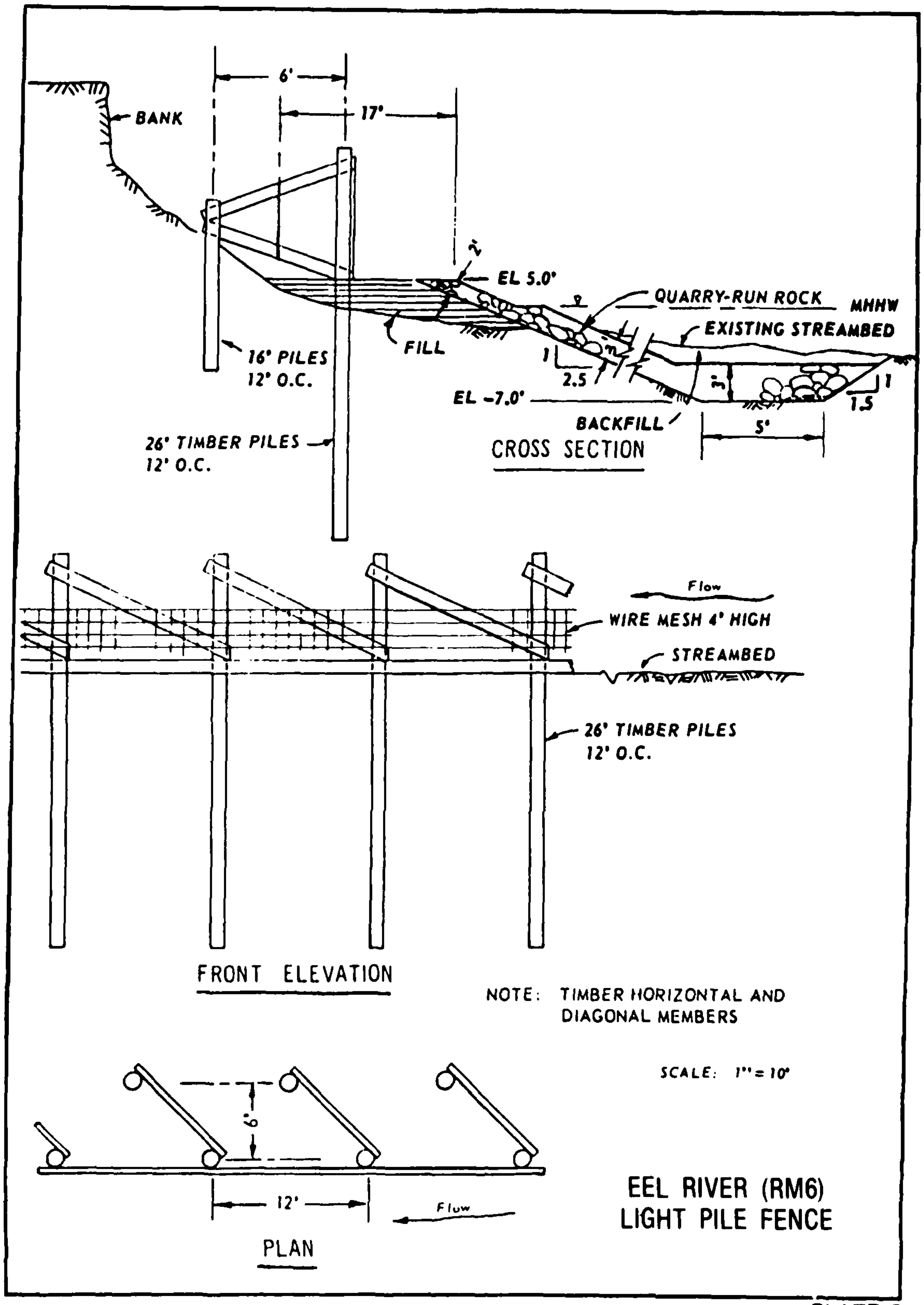

BANK
6'
17'
2'
EL 5.0'
QUARRY-RUN ROCK
MHHW
EXISTING STREAMBED
FILL
16' PILES
12' O.C.
1
2.5
3'
1
1.5
EL -7.0'
BACKFILL
5'
26' TIMBER PILES
12' O.C.
CROSS SECTION
Flow
WIRE MESH 4' HIGH
STREAMBED
26' TIMBER PILES
12' O.C.
FRONT ELEVATION
NOTE: TIMBER HORIZONTAL AND
DIAGONAL MEMBERS
SCALE: 1" = 10'
6'
12'
Flow
PLAN
EEL RIVER (RM6)
LIGHT PILE FENCE

PLATE 5

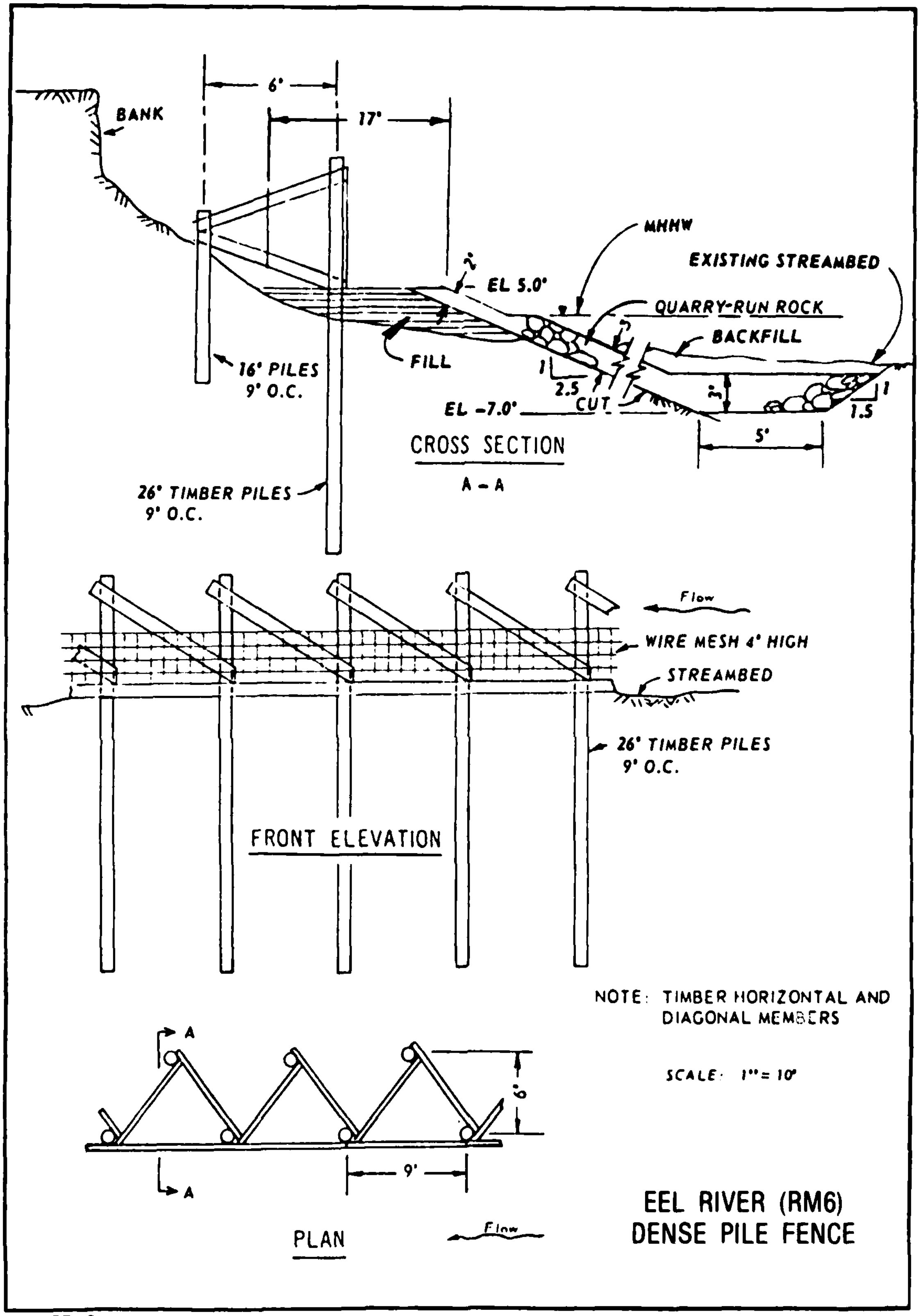
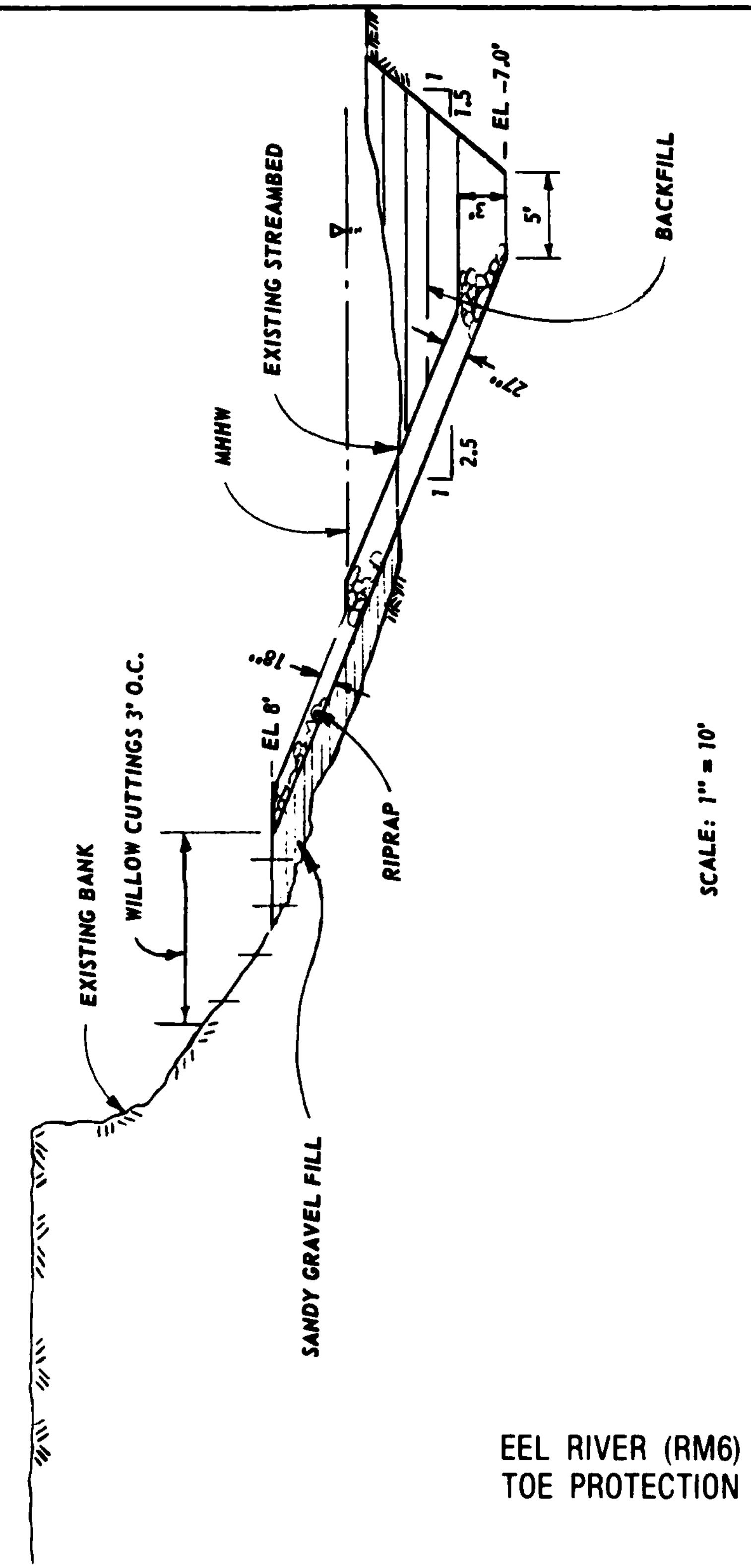
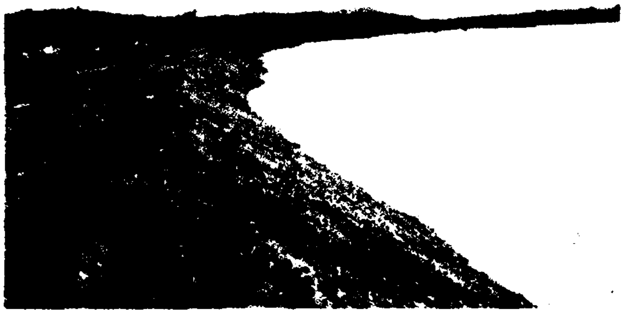

6'
17'
BANK
MHHW
EXISTING STREAMBED
EL 5.0'
QUARRY-RUN ROCK
BACKFILL
FILL
16' PILES
9' O.C.
2.5
CUT
EL -7.0'
1.5
5'
CROSS SECTION
A - A
26' TIMBER PILES
9' O.C.
Flow
WIRE MESH 4' HIGH
STREAMBED
26' TIMBER PILES
9' O.C.
FRONT ELEVATION
NOTE: TIMBER HORIZONTAL AND
DIAGONAL MEMBERS
SCALE: 1" = 10'
A
6'
9'
A
PLAN
Flow
EEL RIVER (RM6)
DENSE PILE FENCE

PLATE 6

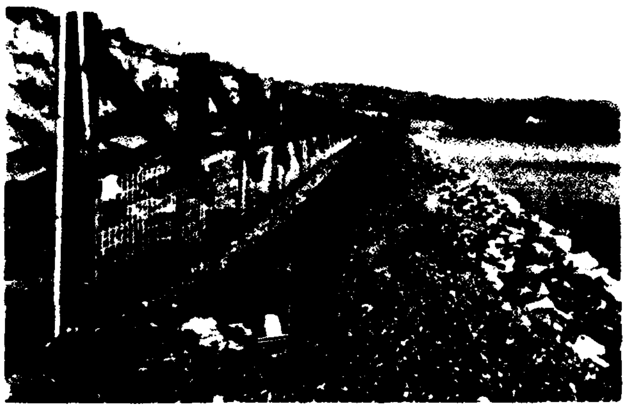


EEL RIVER (RM6)
TOE PROTECTION

PLATE 7



HARDPOINT REACH



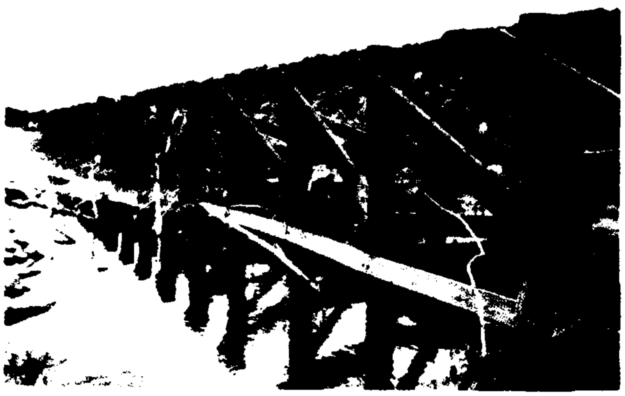
TIMBER PILE REACH

ROCK TOE REACH

EEL RIVER (RM6)
POST-PROJECT CONDITIONS

PLATE 8

HARDPOINT REACH

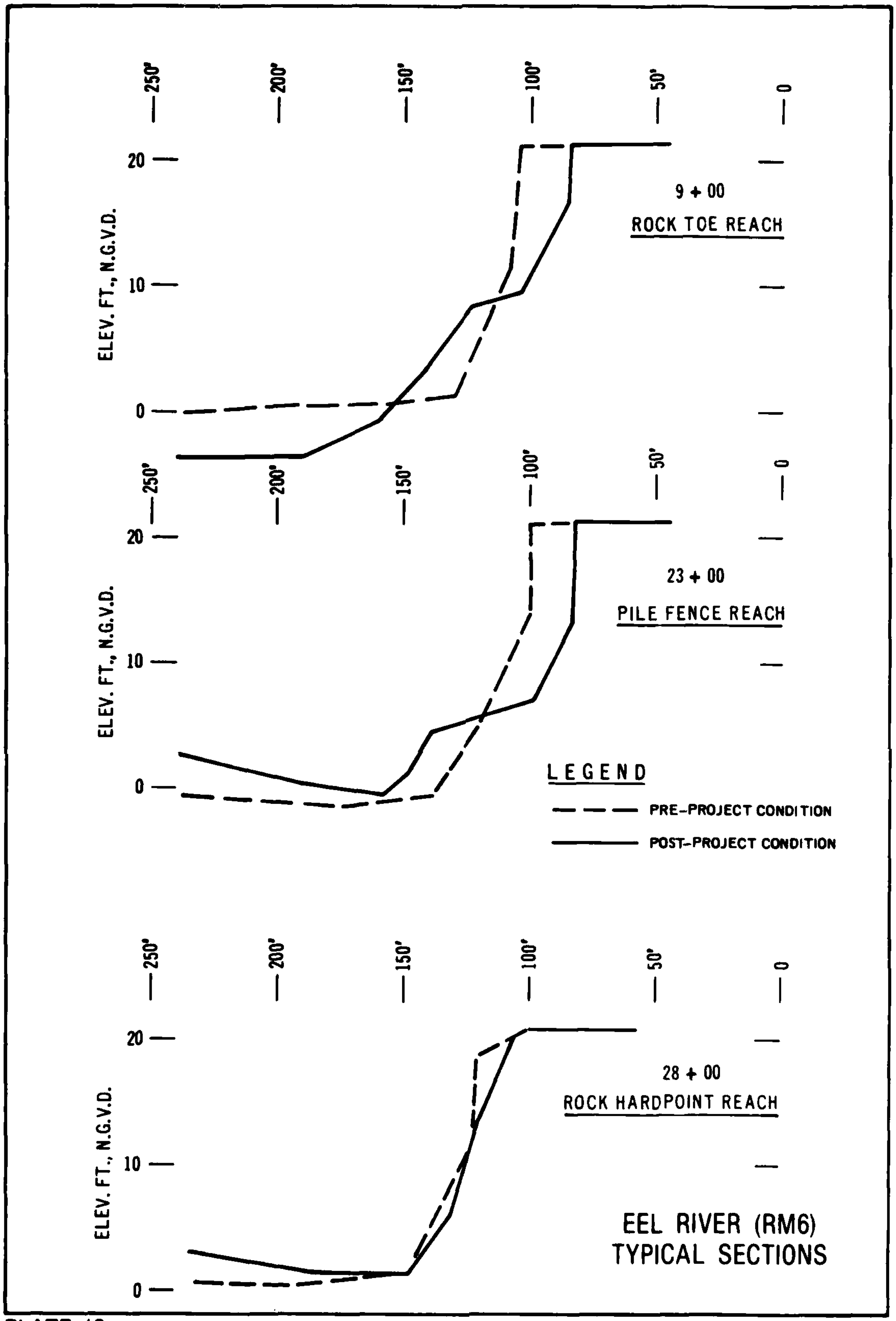


TIMBER PILE REACH

ROCK TOE REACH

EEL RIVER (RM6)
POST PEAK FLOW CONDITIONS

PLATE 9



9 + 00
ROCK TOE REACH
23 + 00
PILE FENCE REACH
28 + 00
ROCK HARDPOINT REACH
ELEV. FT., N.G.V.D.
250'
200'
150'
100'
50'
0
20
10
0
LEGEND
PRE-PROJECT CONDITION
POST-PROJECT CONDITION
EEL RIVER (RM6)
TYPICAL SECTIONS

PLATE 10

Task	Executor of Task: District Office	Executor of Task: Eureka Project Office	Executor of Task: Voluntary Observer(s)	Executor of Task: Other	Performance Period: Prior to Construction	Performance Period: During Construction	Performance Period: After Construction	Post-Construction Data Collection Period
1. Climatology								
a. General				X*	X	X	X	Hourly
b. Raingage			X				X	Daily
2. Hydrology								
a. Discharge				X**	X	X	X	Continuous
b. Crest-Stage Gage	X	X					X	6/Year
c. Staff Gages (2)	X	X	X				X	As Necessary
d. Current Velocities	X	X			X		X	6/Year
3. Surveys								
a. Cross Sections	X	X			X		X	Annually
4. Soils and Geology								
a. Subsurface	X				X			
b. Piezometers (2)	X	X			X	X	X	8/Year
5. Environment								
a. Assessment	X				X			
b. Observations	X	X			X	X	X	As Necessary
6. Project Performance								
a. Erosion	X	X			X		X	Annually
b. Structures	X	X					X	Annually
c. Photographs								
(1) Ground	X	X			X	X	X	8/Year
(2) Aerial	X				X		X	Variable

* National Weather Service ** U. S. Geological Survey

EEL RIVER
MONITORING PROGRAM

PLATE 11

VAN DUZEN RIVER NEAR
CARLOTTA, CALIFORNIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

VAN DUZEN RIVER NEAR CARLOTTA, CALIFORNIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location: The 900-feet long Van Duzén River Demonstration Project for Streambank Erosion Control is located on the right bank (north side) of the river, at River Mile 8, about two miles east of Carlotta, California, as shown on Plate 1.

2. Authority: The Van Duzen River Demonstration Project evolved from the Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251, as amended by Section 155 of the Water Resources Development Act of 1976. This demonstration project constitutes a part of the national program to develop, construct and evaluate innovative, unproven methods of streambank erosion control.

3. Purpose and Scope: This report describes the bank erosion problem, the types of protection designed and constructed, and the performance of the project, which was monitored by the San Francisco District.

4. Problem Resume: The demonstration project is located just downstream of Fielder Creek, a small tributary to the Van Duzen River, where the river has caused serious bank erosion and has threatened several homes in the vicinity. Erosion problems are the

result of high velocities in a relatively steep gradient stream. The river banks at the site extend about 15 feet above the low-flow water surface with the upper 5 feet near vertical and the lower talus slope averaging 1V on 1.5H. The river bank is predominantly silt with some sand and gravel while the streambed is composed of sand, gravel and cobbles.

II. HISTORICAL DESCRIPTION

5. Stream

a. Topography: The Van Duzen River is about 73.5 miles in length with elevations ranging from 100 feet to about 5,200 feet.* Throughout their lengths, the Van Duzen River and its tributaries flow through narrow, steep-sided canyons. The average slope of the river is about 11 feet per mile.

b. Geology: The Van Duzen River basin is underlain by late Jurassic to late Cretaceous rocks of the Franciscan assemblage. This assemblage principally consists of Graywacke sandstones containing subordinate amounts of shale and conglomerate, interlayered with submarine volcanic rocks of extrusive origin and serpentine. All of the Franciscan rocks have been subjected to several episodes of intense folding and faulting during the geologic past. Isolated patches of Tertiary rocks are found in down-faulted blocks near ridge crests in the upper basin area.

c. Climatological Characteristics: The climate of the basin is generally mild; summers are warm and dry. Only about 5 percent of the normal annual rainfall occurs from June to September whereas about 35-40 percent of the normal annual rainfall occurs in December

---

* All elevations mentioned in this report refer to National Geodetic Vertical Datum unless otherwise noted.

and January. Climatological data are available from National Weather Service stations at the following locations:

Station	Data
Bridgeville 4 NNW	daily rainfall
Bridgeville 5 W	hourly rainfall
Eureka WSO CI	hourly & daily rainfall, wind
Kneeland 10 SSE	hourly rainfall
Scotia	daily rainfall

d. Hydrologic Characteristics: Streamflow data for the Van Duzen River were developed for the report "Flood Plain Information, Van Duzen River, Humboldt County, California," dated July 1973. Discharges adjacent to the project site represent runoff from a drainage area of about 275 sq mi. The peak discharge vs frequency relationship is presented in the following tabulation:

	Peak Discharges in Cubic Feet Per Second	
Event	Van Duzen River near Bridgeville	Van Duzen River at Project area
100-Year	54,000	60,000
50-Year	49,000	54,000
20-Year	41,000	45,000
10-Year	35,000	39,000
5-Year	29,000	33,000
2-Year	21,000	23,000

The Standard Project Flood (SPF) near Bridgeville and in the vicinity of the project area is estimated to be 68,000 and 75,000 cubic feet per second (c.f.s.), respectively.

Discharge vs. stage relationships developed during the flood plain information study are augmented by topographic and hydraulic data collected during the monitoring period. The following discharge vs. stage relationship exists in the project area:

Peak Discharge (cubic feet per second)	Stage * (feet)
0	0.0
5,000	6.5
10,000	9.7
20,000	13.4
30,000	15.0 **
50,000	17.0
54,000 (Dec 1964)	17.3
60,000	17.8
70,000	18.9
75,000 (SPF)	19.1

* From streambed.
** Approximate channel capacity.

e. Stream and Stream Characteristics: The Van Duzen River is a principal tributary to the Eel River. The Van Duzen River has a streambed slope of about 11 feet per mile. The basin drainage pattern is principally trellis with streams flowing through narrow, steep-walled, V-shaped canyons.

f. Environmental Considerations: The environmental aspects of the project, including water quality, vegetation, fish and wildlife, benthos, rare and endangered species and cultural resources, were carefully reviewed and coordinated prior to the construction of the project. The proposed project was reviewed by the U.S. Fish and

Wildlife Service, the State Department of Fish and Game, and the North Coast Region of the California Regional Water Quality Control Board.

An environmental asessment was prepared in accordance with Section 404(b) of Public Law 92-500. In addition, a public notice describing the project distributed to about 125 Federal, State and Local agencies and property owners.

In accordance with the requirements of the National Environmental Policy Act of 1969 (Public Law 91-190), the Corps of Engineers has evaluated the environmental aspects of the proposed activity. From an analysis of these impacts, was determined that the activity would have no significant effect on the quality of the environment. Therefore, an Environmental Impact Statement was not prepared.

6. Demonstration Test Site Reach

a. Hydraulics & Hydrology. The slope of the Van Duzen River at the project site is about 11 feet per mile. Most of the rainfall occurs during the months of December and January. Flows in the river have varied from an minimum of 10 c.f.s. in the dry season to a maximum of about 54,000 c.f.s. which occurred during the December 1964 flood. The 1964 flood was about 3 ft over the top of bank. It is estimated that the maximum velocities along the front of the works will approximate 10 feet per second (f.p.s.) and that velocities through and behind the works will reach 5 f.p.s. during extreme events such as the December 1964 flood. The channel capacity is estimated to be about 30,000 c.f.s, which is approximately a 5-year event.

b. River bank Description

1. Bank Material. The bank along the demonstration site is about 15 feet in height, and consists primarily of silt and gravel. The upper 5 feet is near vertical and the lower talus slope is about 1V on 1.5H. The riverbed consists of sand, gravel and cobbles; boulders up to 10 inches in size can be found in the riverbed.

2. Normal Bank Vegetation. Prior to the demonstration project, the vegetative cover on the banks at the site consisted of grasses and occasional willows. The high velocities of the stream impacting directly on the highly erosive bank material accounted for the sparse vegetative cover.

3. Bank Erosion Tendencies. The banks along the test site have been eroding at varying rates. Significant erosion has occurred when the stage in the river approached the upper 5 feet of streambank along the test site. Pictures of the erosion problem at the site are shown on Plate 2. Plate 3 shows the demonstration site after two flood seasons. The demonstration project site is on the outside of a curve with a radius of about 1,000 feet and the average bankfull width is about 1,000 feet. The river flows between its banks, attacking the outside of bends and building up bars on the inside of bends.

III. DESIGN AND CONSTRUCTION

7. General: This project was formulated on the theme of testing locally innovative, environmentally acceptable methods of controlling streambank erosion. In lieu of the more conventional riprapping of the sideslopes, methods considered compatible with the type of streamflow and streambank conditions, and which used locally readily available materials, were considered for the Van Duzen River

demonstration project. The adopted methods were: Tree Pendants and two variations of Timber Pile Fence.

8. Basis for Design: The design was based upon engineering judgment, observed effectiveness of comparible existing bank protection measures, inspection of the site, topographic surveys, soils and foundations investigations and an estimate of forces that could reasonably be imposed upon the component parts of the structure. Improvements were designed to provide protection during a 10-year event (39,000 c.f.s.) in the river. The following paragraphs present design details for the different types of bank protection works which were designed and constructed; the location of each is shown on Plate 4.

a. Tree pendants with anchors and ties were designed for forces produced by flotation, flowing river currents, and the impact of floating debris. The anchors and ties were designed for a velocity of eight feet per second (fps) along the river face of the pendants and five fps through the pendants. Consideration was given to floating debris impacting the trees and some debris collecting within the line of tree pendants. Driven Douglas fir wood pile anchors were designed for a compression parallel to grain strength of 1,250 pounds per square inch (psi) and a bending strength of 2,450 psi. The tree pendants were tied together with 3/8-inch wire rope to help maintain an effective alignment during high flows.

b. The pile fence was designed for forces produced by the tendency for the wood piles and wood bracing to float, the flowing river currents, and the impact of floating debris. It was estimated that the maximum velocity along the river face of the structure would be 8 fps and through the structure would be 5 fps. The wire fence along the channel face of the structure would tend to collect debris and reduce and deflect river currents along the fence. The chance of direct impact by floating debris would be reduced by the

wire fence. Consequently, the piles and wood bracing were designed for glancing blows, not direct impact by large floating debris in the river. It was assumed that the maximum impact force from floating debris would be equivalent to the force produced by a floating log 2 feet in diameter, 25-feet long, and moving at 5 fps. The Douglas fir wood bracing was designed for a compression parallel to grain strength of 1,250 psi and a bending strength of 1,500 psi. The wood piles were designed for the same unit working stresses as presented above for anchorage of the tree pendants. The allowable skin friction force for the wood piles was determined to be about 1,500 psi from the soils and foundation investigations. The four-foot high wire fence consisted of heavy galvanized field fencing with widely spaced wires for the downstream 300 feet and six-inch square welded plain wire mesh for the upstream 300 feet.

9. Construction Details

Two different types of bank protection were used at the project site as shown on Plate 4. The total project length was 900 feet. The upstream 300 feet consisted of tree pendants and the downstream 600 feet consisted of 300 feet of light pile fence and 300 feet of dense pile fence. A general description of each type of streambank protection used is contained in the following paragraphs:

a. Tree Pendants: The upstream 300 feet of the project consists of tree pendants, shown on Plate 5. Freshly cut, whole trees, 40 to 50 feet high were placed in an overlapping pattern along the face of the eroding bank. Each tree was anchored to a deadman and was tied to adjacent pendants by screw anchors.

b. Light Pile Fence: The middle 300-foot section of the project consists of light timber piles, 20 feet in length, equipped with driving shoes, driven at least 11 feet into the ground along the bank at 12-foot centers, and faced with timber bracing and wire

mesh. The douglas fir piles, which were debarked and pressure treated, had a minimum tip diameter of 8 inches and a minimum butt diameter of 12 inches. Each pile was tied to adjacent piles with timber members as shown on Plate 6. Timber members varying from two inches by 10 inches to three inches by 12 inches were used to transmit impact loads to a second row of piles located six feet behind (landward) of the 20-foot long piles. The second row of piles were 10 feet in length, equipped with driving shoes, and driven at least 6 feet into the ground. Each set of piles was tied together by one horizontal and one diagonal timber member to resist impact forces. Galvanized 3/4-inch steel bolts, washers and nuts were used to connect the piles to the timber bracing In order to reduce velocities and encourage sedimentation behind the piles, a 4-foot high wire mesh fence was attached to the bottom of the outer row of piles.

c. Dense Pile Fence: The dense pile fence, as shown on Plate 7, was a modification of the light pile fence, and was built for a length of 300 feet downstream of the light pile fence. Piles were driven at 9-foot centers. They were tied to the back row of piles, however, in both the downstream and upstream directions. It was believed that it created a stronger structure and would encourage more sediments to be deposited behind the pile fence by the additional resistance to flow created by the triangular-shaped bracing pattern. Six-inch square welded wire mesh was attached to the bottom of the outer row of piles to reduce velocities and encourage sediment deposition behind the piles. All other aspects of this reach of fence were the same as that described for the light pile fence.

10. Cost: The total cost of construction for the three types of bank protection was $138,900. Included in the total construction cost is about $32,900 for engineering and design, and $19,300 for supervision and administration (E&D and S&A). The total construction cost associated with each type of streambank protection

employed at this demonstration site are as follows:

Method of Protection	Cost Per Foot (Including E&D plus S&A)	Cost Per Foot (Contract Cost Only)
Tree Pendants	$ 73.00	$ 15.00
Light Pile Fence	$168.00	$110.00
Dense Pile Fence	$222.00	$164.00

11. Monitoring Program: The monitoring program set forth in this report was developed in accordance with guidances provided in the Waterways Experiment Station repo "Guidelines for Monitoring and Reporting Demonstration Projects", dated September 1977, and in accordance with discussions with the South Pacific Division office representative on the Section 32 Steering Committee. The goal of the monitoring program was to augment existing data and data collection programs to the extent necessary to monitor factors which may have an effect on the erosion problems at the site. The extent of the monitoring program is summarized on Plate 8.

A chronological listing of significant events occurring at the site since construction are as follows:

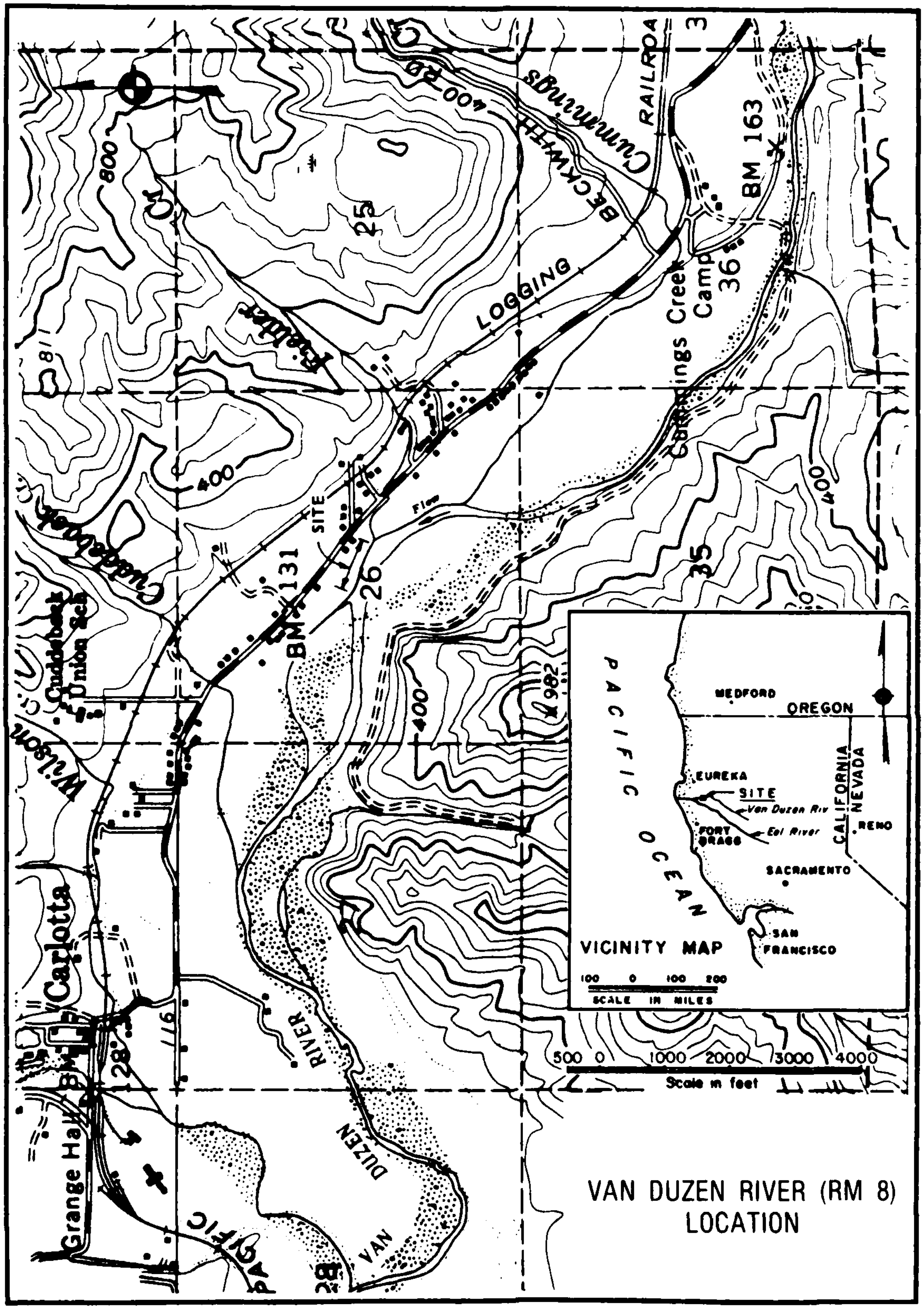
Date	Description
11/78	Construction Completed.
12/78	Monitoring of Project Underway.
12/78	Installation of Staff and Crest Gages.
11/79	Site Inspection by H & H Section. No significant flows experienced.

	Site was performing satisfactorily. High water evidence to within 1 foot of the top of the wire fence.
1/80	Three-day storm. Runoff waters rose about 4 feet on the pile fence. No damage sustained by the test site.
1/81	No appreciable runoff during current flood season.

12. Evaluation of Protection Performance: The demonstration project has performed in a satisfactory manner. Although the tree pendants have performed satisfactorily for the flows which have been experienced at the site, it should be noted that the trees as placed did not meet the real intent as envisioned during the design of the project. The trees used were Redwood trees, which were uprooted from an area upstream of, and across the river from, the demonstration project. During the falling and moving of the trees to the project site, substantial portions of the root system and many of the limbs broke off the trees. Additional limbs were broken during placement, such that only the trunk and a few limbs remained. It is believed that the redwood trees used were too brittle to perform as intended and may have, in fact, been less than completely healthy. It is suggested that care be taken to select trees which are not brittle and which are healthy for this type of application. Although it was hoped that higher discharges could be experienced, to severely test the structure, the flows to date have promoted the anticipated deposition of debris with the proliferation of voluntary willow growth. If another low-flood year is experienced this winter season, it is reasonable to expect the willow growth will become firmly established. No measurable amount of streambank has been

lost since completion of the demonstration site.

13. Conclusion: The overall performance of the demonstration site to date can be considered successful. No damage has occurred in the entire test reach. At the time of project formulation, it was observed that a land slide had occurred and was encroaching on the left bank of the channel a short distance upstream of the selected project site. This caused flood flows to be diverted toward the project area. Over the past two runoff seasons, it has been observed that most of the slide has eroded and that the primary river flow has been re-oriented away from the demonstration site. Therefore flows directed toward the demonstration site are presently overbank or secondary type flows. Extensive willow growth and silt deposition is expected to continue and promote control of streambank erosion.



VAN DUZEN RIVER (RM 8)
LOCATION

PLATE 1

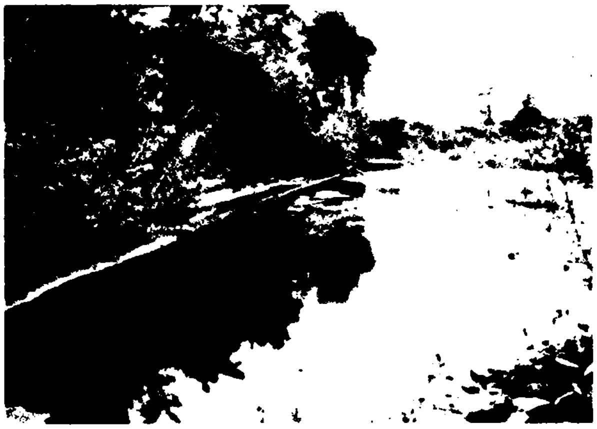
PROJECT SITE

ERODED BANK, TYPICAL

LOWER END OF PROJECT SITE

VAN DUZEN RIVER (RM 8)
PRE-PROJECT CONDITIONS

PLATE 2



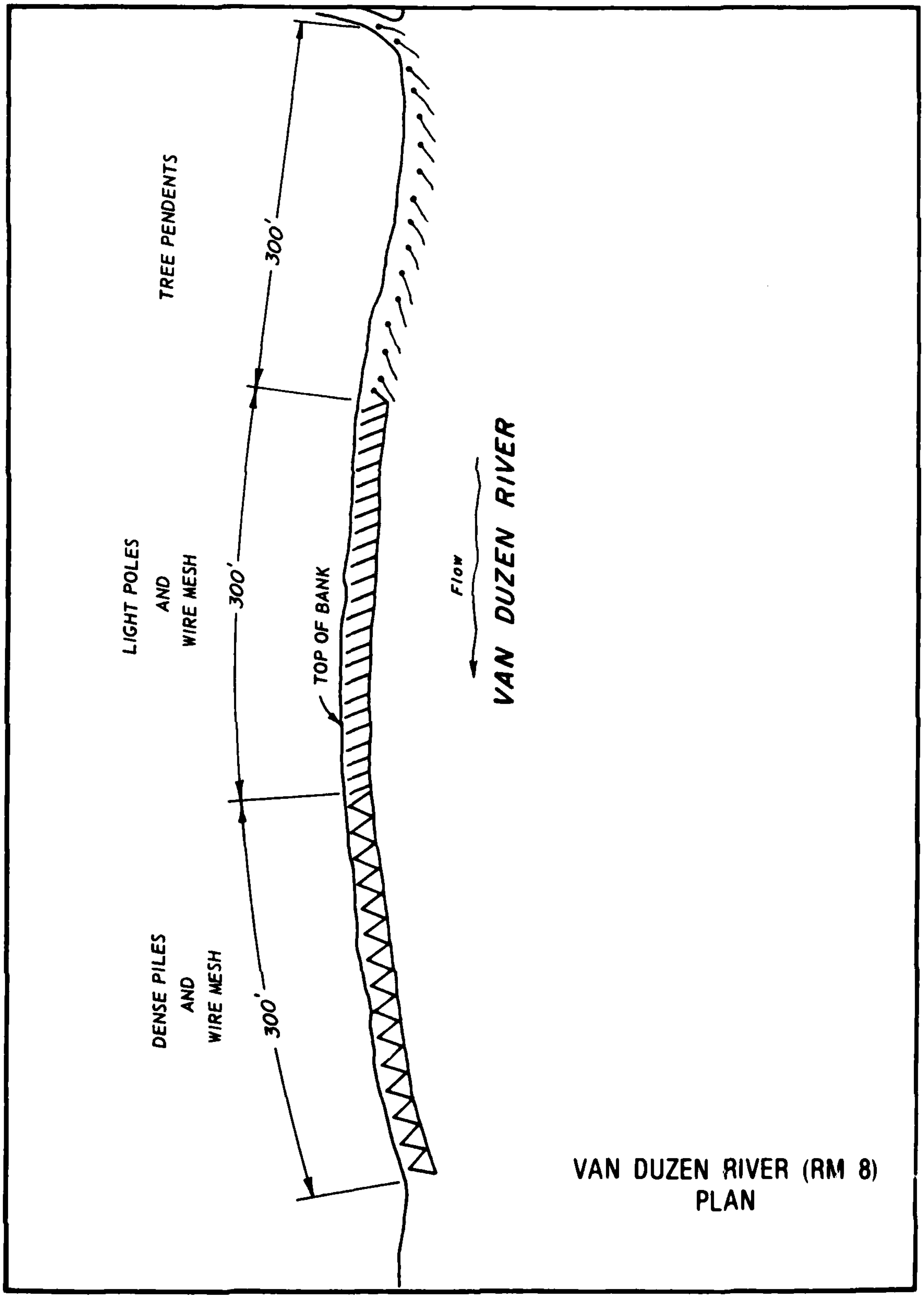
TREE PENDANT REACH DURING
SECOND YEAR OF TESTING

PILE FENCE REACH SHORTLY AFTER
COMPLETION OF CONSTRUCTION

PILE FENCE REACH DURING
SECOND YEAR OF TESTING

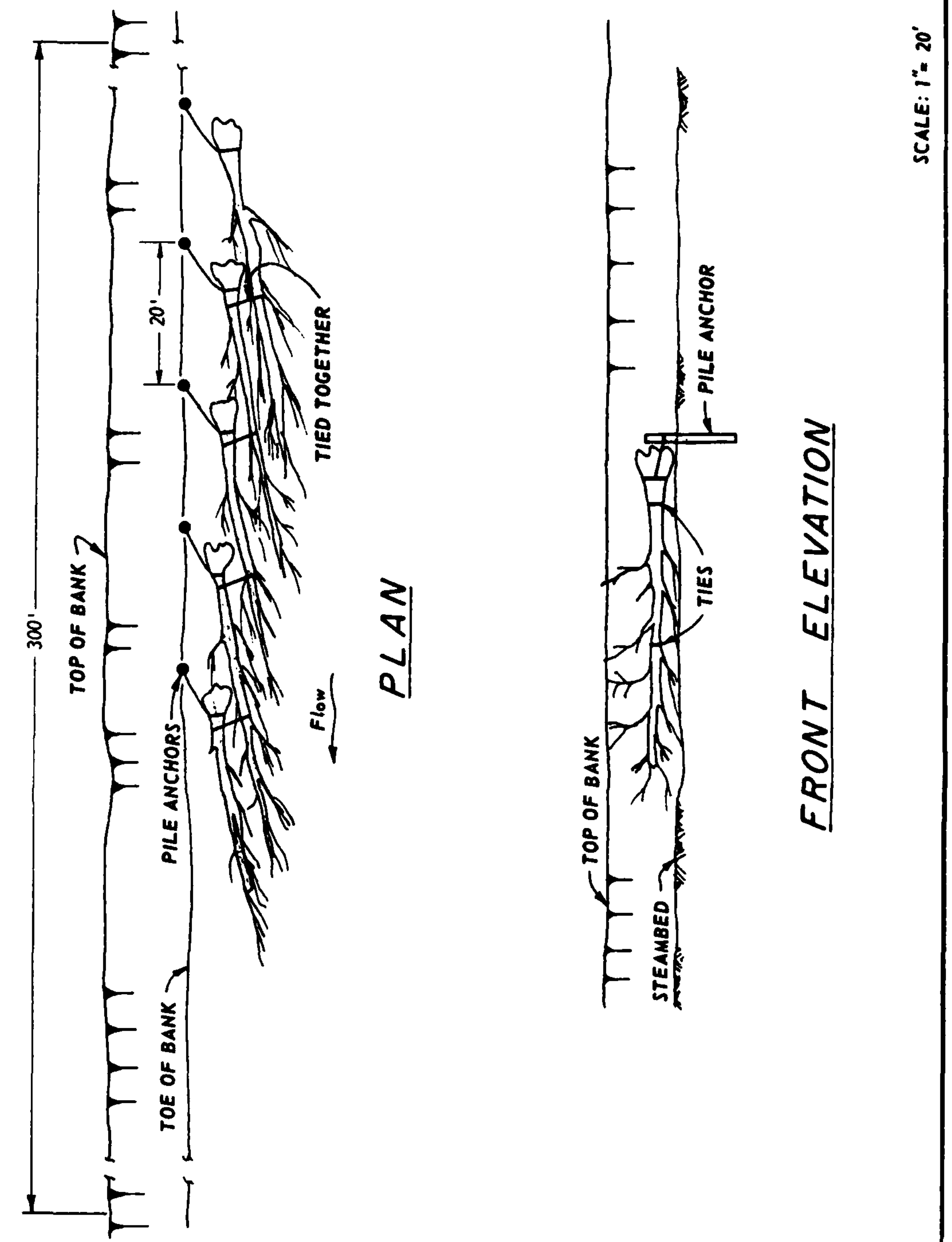
VAN DUZEN RIVER (RM 8)
POST-PROJECT CONDITIONS

PLATE 3



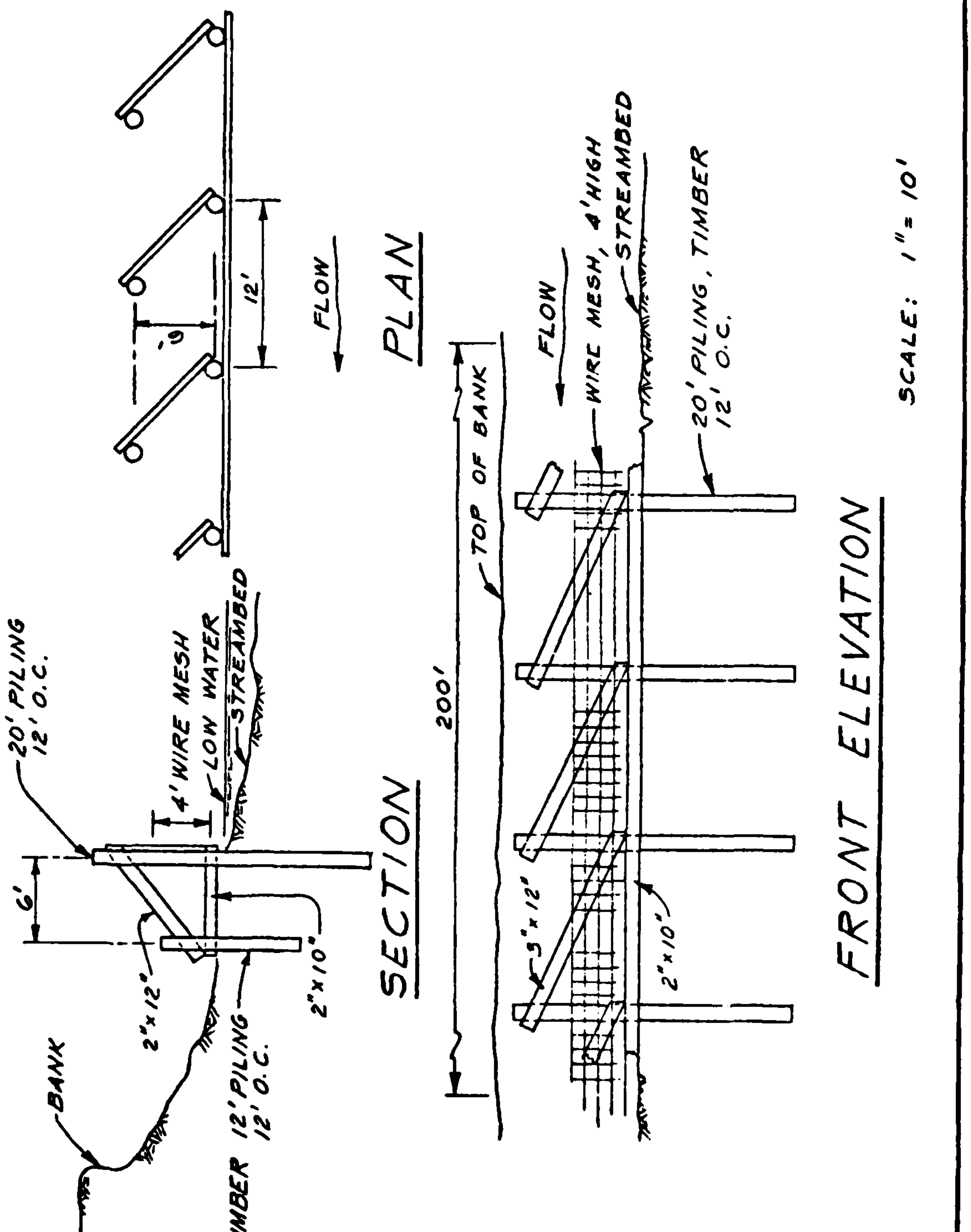
VAN DUZEN RIVER (RM 8)
PLAN

PLATE 4



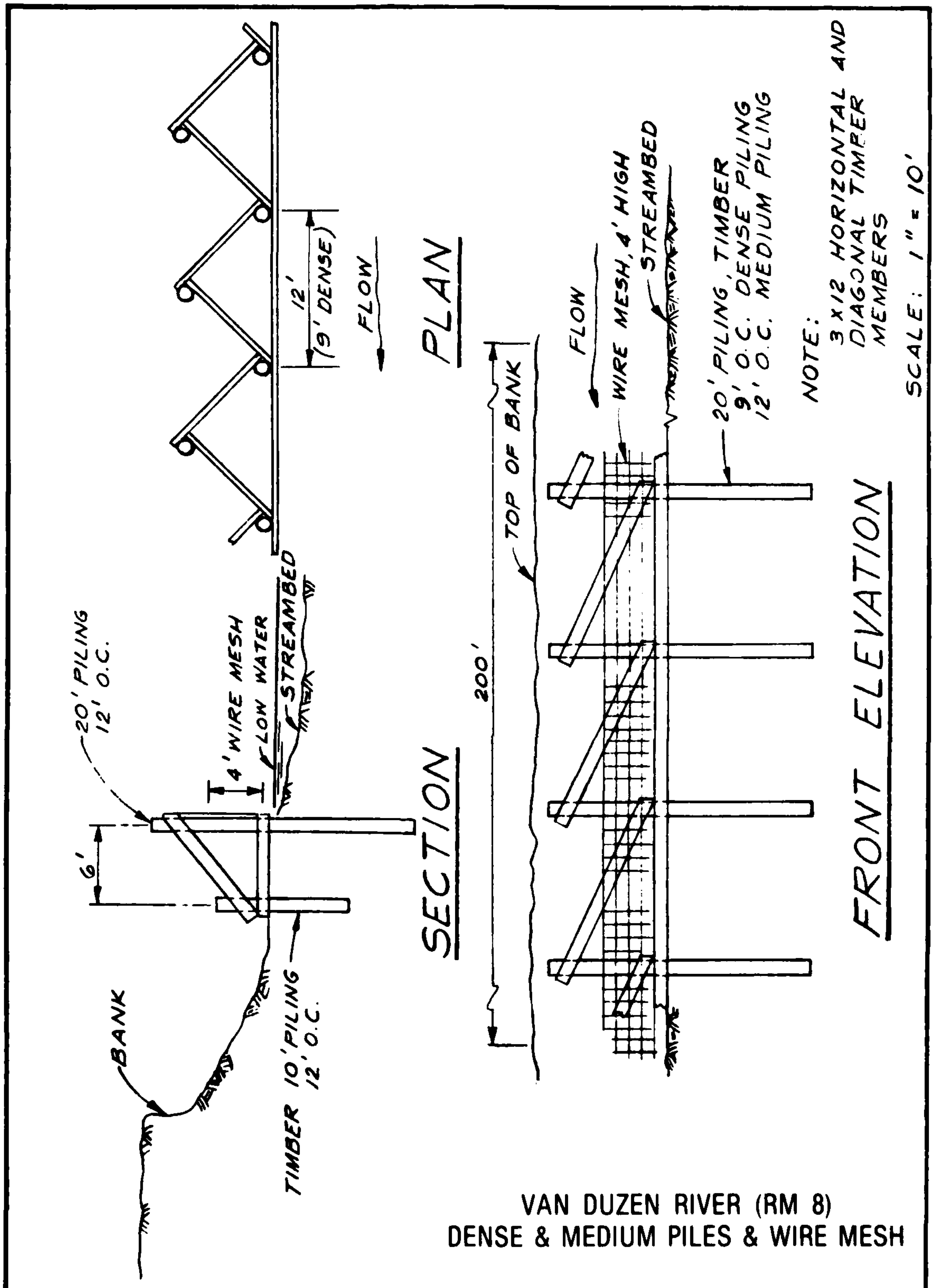
VAN DUZEN RIVER (RM 8)
TREE PENDANTS

PLATE 5



VAN DUZEN RIVER (RM 8)
LIGHT PILE FENCE

PLATE 6



VAN DUZEN RIVER (RM 8)
DENSE & MEDIUM PILES & WIRE MESH

PLATE 7

Task	Executor of Task: District Office	Executor of Task: Eureka Project Office	Executor of Task: Property Owner	Executor of Task: Other	Performance Period: Prior to Construction	Performance Period: During Construction	Performance Period: After Construction	Post-Construction Data Collection Period
1. Climatology								
a. General				X*	X	X	X	Hourly
b. Raingage			X				X	Daily
2. Hydrology								
a. Discharge				X**	X	X	X	Continuous
b. Crest-Stage Gage	X	X					X	6/Year
c. Staff Gages (2)	X	X	X				X	As Necessary
d. Current Velocities	X	X			X		X	6/Year
3. Surveys								
a. Cross Sections	X	X			X		X	Annually
4. Soils and Geology								
a. Subsurface	X				X			
b. Piezometers (2)	X	X			X	X	X	8/Year
5. Environment								
a. Assessment	X				X			
b. Observations	X	X			X	X	X	As Necessary
6. Project Performance								
a. Erosion	X	X			X		X	Annually
b. Structures	X	X					X	Annually
c. Photographs								
(1) Ground	X	X			X	X	X	8/Year
(2) Aerial	X				X		X	Variable

* National Weather Service ** U. S. Geological Survey

VAN DUZEN RIVER
MONITORING PROGRAM

PLATE 8

ALLEGHENY RIVER NEAR
WATTERSONVILLE, PENNSYLVANIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

ALLEGHENY RIVER NEAR WATTERSONVILLE, PENNSYLVANIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

A. Project Name and Location Wattersonville, Pennsylvania, Demonstration Project, Allegheny River - mile 62.5, one and one-half miles downstream of Wattersonville, Pa. and immediately upstream of Lock and Dam No. 9. Plate 1 shows the project location.

B. Authority Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, P.L. 93-251.

C. Purpose and Scope This report describes a bank erosion problem, the types of protection used, and a performance evaluation of a demonstration project on the Allegheny River designed and monitored by the Pittsburgh District.

D. Problem Resume The right bank of the Allegheny River was subject to active erosion which was encroaching on mature woodland and land used for seasonal and permanent residences. The property owners attempted to protect the bank with a variety of schemes with varying degrees of success. The local protection efforts were constrained by the narrow scope of each owner's concern and the limited resources available.

II. HISTORICAL DESCRIPTION

A. Stream Description, General

1. Topography

The demonstration site is located in Armstrong County,

approximately 1-3/4 miles south of the confluence of Red Bank Creek with the Allegheny River and imemdiately north of Lock and Dam No. 9 on the right bank of the Allegheny River, in the East Brady 7-1/2 minute quadrangle. This area is located entirely within the Allegheny Plateau province and is characterized by relatively flat lying strata which are broken only by small faults and low broad folds. The topography consists of steep sided, narrow stream valleys away from which the relief becomes more uniform with low rolling hills. Elevations range from approximately 840' near the Allegheny River Bank to 1500' at some of the hill crests. The Allegheny River is the principal stream of the quandrangle and flows in a southeasterly direction. Within the area of the demonstration site, the riverbed gradient is 0.62 feet per mile while the water surface gradient is about 1.0 feet per mile. The Allegheny River is the main headwater tributary to the Ohio River draining approximately 11,500 square miles. Of this total drainage some 8400 sq. miles are located above the demonstration site. Red Bank Creek is a principal tributary to the Allegheny and is the second most important stream in the area having a total length of about 75 miles.

2. Geology

The demonstration site is situated on the northwest flank of the Kellersburgh anticline, a southwest-northeast trending anticline with its axis plunging to the southwest. The dip of the folded strata is low with a maximum of only 3 to 4 degrees. The Kellersburg anticline is the main structural feature of the quadrangle and is bounded to the northwest by the Brady's Bend syncline. The structures were folded at the same time as the Appalachian Mountains but were the result of weaker stresses.

The project area is located on the flood plain of the Allegheny River. The associated stream deposits are alluvium consisting of an unconsolidated, heterogenous mixture of clay,

silt, sand and boulders. The deposits are predominately recent in age although in other parts of the quadrangle some deposits date back to the Pliestocene and consist of glacial gravels of the Kansan glaciation. These glacial gravels are the result of outwash and probably have been reworked many times since the East Brady quadrangle itself is not glaciated.

The bedrock exposed or immediately beneath the demonstration site is a series of alternating marine and non-marine deposits consisting of sandstone, shale, limestone, and coal. These sedimentary rocks were deposited on or a short distance seaward from a large delta that advanced westward during the time of the Pennsylvanian coal swamps.

3. Locality, Development, and Occupation

All the land upstream of federal property at the project site is privately owned and a good portion is partially cleared and improved with residential structures which are used as second homes. The major land uses in northern Armstrong County are forestland, 61%; cropland/pasture, 26%; extractive mining, 10%; and residential, 2%. The amount of industry in this area is negligible, occupying only 0.4% of the land.

The presence of second homes in this area is indicative of the seasonal and recreational use of this section of the Allegheny River. In 1974, Lock and Dam No. 9 recorded 30,700 recreation days of use in the form of boating, fishing, waterskiing and picnicking. This figure, however, fails to include river traffic not using the lock facilities, so actual river recreational usage is probably considerably higher. In contrast, no commercial lock traffic at Lock and Dam No. 9 and Lock and Dam No. 8 was reported for 1974 and it continues to remain very low.

The downstream end of the demonstration project contacts the right abutment of Lock and Dam No. 9. The lock is sited along the left bank, opposite the demonstration project. Lock

and Dam No. 9 began operation in October 1938 and lifts river traffic 22 feet in a 56 foot wide by 360 foot long chamber. The dam is a 950 foot long fixed concrete weir.

4. Hydrologic Characteristics

The climate is humid continental with a normal seasonal variation, and is subject to rapid changes due to frontal air mass movements. Winds are generally light to moderate, averaging nine miles per hour year-round, with greater average velocities occurring during winter months. Wind direction is modified by topographic features, but generally ranges from a northwesterly to southwesterly direction. Precipitation is distributed evenly throughout the year averaging about 41 inches total. The annual average temperature is about 48°F with average monthly temperatures of 25°F in January and 70°F in July. The flood of record occurred in January 1959 with a maximum discharge of 205,000 c.f.s. at the demonstration site. A stage-frequency curve and a one-year flood hydrograph are shown on Plate 2.

5. Existing Channel Conditions

The demonstration project is located on a south flowing reach of river which is tangent to the downstream end of a 1.2 mile radius curve and the upstream end of a 1.4 mile radius curve. A discharge rating curve for the river at the site location is shown on Plate 3.

6. Environmental Considerations

The slopes and uplands adjacent to the river support a tuliptree-red oak-sugar maple community with lesser amounts of ash, black walnut, white pine, Virginia pine, black oak and rhododendron. In general, this is a mature stand of forest with the diameters of some trees ranging from one to three feet.

Wildlife associated with this area and vegetation include woodcock, black bear, ruffed grouse, white-tailed deer, gray and fox squirrels, cottontail rabbits, gray fox, racoons and woodchucks. Gamefish in this river section include muskellunge, northern pike, large and smallmouth bass, walleye and other warm water fish.

The Carnegie Museum of Natural History conducted an archaeological survey of the demonstration site and found artifacts which indicated that the lower river terraces were used by the Middle to Late Woodland people and that the higher terraces were used by the Archaic cultures. The survey concluded that the project would cause minimal loss of archaeological resources and would help protect remaining resources from loss due to erosion.

7. Environmental Effects.

The proposed schemes should alleviate the erosion problem at this location and the corresponding loss of riverbank and riparian vegetation, and siltation of the river. To do this however, construction activities over an approximate 90-day period will disturb the littoral and riparian zones resulting in temporary and localized turbidity and siltation, disturbance of resident wildlife populations, and a permanent, but minor, replacement of littoral and riparian habitat with stone structures. The long range impact on water quality, the flora, and fauna of this area will be negligible.

One or more optimum designs of bank protection will be determined as a result of this study which will be of great value for future erosion control projects along the Allegheny River. At the project site itself, the proposed schemes should collectively relieve the acute bank erosion problem which has caused loss of private property and threatens to undercut the foundations of some privately owned structures. The erosion problem also degrades the aesthetic appeal of this area for

recreational users, and while construction activities will cause a temporary aesthetic disturbance and the finished project will not restore a natural riparian setting, the overall aesthetic appeal of this area should not be further degraded by the implementation of this project.

B. Demonstration Project

1. Hydrologic Characteristics

The hydrograph for 1980 for the demonstration site is shown on Plate 4. Channel cross-sections are shown on Plate 5. Ice develops a significant cover across the full width of the river annually at the demonstration site. Massive ice floes develop as the ice cover breaks and moves downstream in the late winter. Piles of loose ice move along the riverbank gouging soil and damaging the riparian vegetation.

2. Hydraulic Characteristics

River stages at the site are recorded at Lock No. 9, on the opposite riverbank. The minumum pool, elevation 822, is determined by the crest of the fixed weir concrete dam. All flow passes over the dam, thus, the dam influences the upstream river elevation at all stages. Average river velocities at the site for given discharges are as follows:

Discharge cfs	Frequency of Occurrence years	Average Velocity fps
117,500 (1954 flood)	--	4.3
175,000 (1972 flood)	--	5.9
207,700	100	6.6
250,200 (standard project flood)	--	7.6

3. Riverbank Description

The riverbank at the demonstration site is composed of fine grained alluvium deposited by past flood events. The bank varies in height from three to fifteen feet and is disected by

several local drainage swales. The landward area is a mature forest with clearings occupied by private dwellings. The riverbank soils in two typical areas are delineated on Plate 6.

The main cause of bank instability at this site appears to be saturation of the bank during high water and subsequent drawdown which results in stability failures of blocks of soil and in piping cavities formed by concentrations of riverward seepage. High water also erodes bank soil by scour. Erosion due to breakup of winter ice contributes to bank instability at this site.

Erosion of this site was brought to the attention of the District in August 1972, shortly after the tropical storm Agnes flood. The severity of erosion prior to this time was not recorded. There have been no dramatic changes in land use or pool elevation in recent years.

III. DESIGN AND CONSTRUCTION

A. General The Wattersonville site presented the opportunity to evaluate different schemes of bank protection under conditions of severe ice attack as well as warm climate river influences. Five schemes of structural protection were designed to resist the observed mechanisms of erosion.

B. Basis for Design The protective schemes were designed for durability under ice attack, and thus, are more substantial than protection against hydraulic effects alone. Natural limestone was used in lieu of any slag or waste material to avoid degredation of the existing water quality. Stone was used as either graded stone protection, graded in size between 4 and 18 inches, or ungraded stone fill protection with a 24 inch maximum size and a one inch minimum size. Filters of graded sand and gravel as well as plastic filter fabric were used. Vegetal erosion control schemes were not appropriate for use under the severe site conditions.

C. Construction Details

1. Scheme 1

This 800-foot long scheme is subdivided into two variations on a conventional stone revetment. Scheme 1A extends 300 feet upstream from the abutment of Dam No. 9 and comprises a blanket of stone fill with a 2-foot minimum thickness and an outer slope no steeper than one vertical on one and one-half horizontal. The stone fill blanket extends from the top of bank to elevation 822, equal to the dam crest elevation, where the stone fill was built out to form a 3-foot wide horizontal toe berm with a one on one slope from elevation 822 to the existing streambed. Plate 7 shows details of Scheme 1A. Scheme 1B extends 500 feet upstream from Scheme 1A and was constructed by placing a free-draining sand gravel fill to provide a slope no steeper than one vertical on one and one-half horizontal followed by a 6-inch thick graded sand and gravel filter and an 18-inch thick blanket of graded stone protection.

The protection extends from the top of bank to a toe trench excavated to elevation 820, 2 feet below the pool elevation, with a 2-foot bottom width. Scheme 1B includes two concrete sealed gutters to pass local overbank drainage. Scheme 1B terminates upstream in a stone filled cut-off trench excavated 20 feet into the bank face to preclude outflanking by erosion of an adjacent unprotected reach of bank. Plate 8 shows details of Scheme 1B.

2. Scheme 2

This 250-foot long scheme is separated from Scheme 1B by 150 feet of unprotected bank. Scheme 2 was constructed by placing a stone fill dike along the water's edge from elevation 822 to elevation 826 with a 2-foot top width, a one on one landward slope, and a one vertical on one and one-half horizontal riverward slope. A free-draining sand and gravel

fill was placed between the dike and the bank face with a slope of one vertical on three horizontal. The downstream end of Scheme 2 is protected from outflanking with a stone filled cut-off trench. Details of Scheme 2 are shown on Plate 9.

3. Scheme 3

This 320-foot long scheme comprises a stone fill placed against the bank face with a 2-foot width at the top of bank, a one vertical on one and one-half horizontal outer slope, and a toe placed in a trench excavated to elevation 822. The most upstream 100 feet of the scheme include an existing timber retaining wall which was incorporated into the project by placing stone fill against the outer face to elevation 827. The interface between Schemes 2 and 3 includes a stone filled cut-off trench to prevent outflanking if one of the schemes would fail. Plate 10 shows details of Scheme 3.

4. Scheme 4

Scheme 4 comprises seven stone fill hardpoints spaced at 100-foot intervals along a 600-foot reach of bank upstream of Scheme 3. The hardpoints are stone fill dikes, placed perpendicular to the bank, extending 20 to 40 feet riverward from the top of bank and entrenched 10 to 20 feet landward. The hardpoints are intended to protect the bank from ice masses moving longitudinally downstream. Scheme 4A includes the three most downstream hardpoints which protrude riverward to the edge of pool, elevation 822. The remaining four hardpoints in Scheme 4B extend further riverward and offer more substantial resistance to ice floes. Plates 11 and 12 show details of Schemes 4A and 4B.

5. Scheme 5

This scheme is 200 feet long and consists of a stone filled trench constructed parallel to the bank and 6 feet landward.

The trench is 4.5 feet deep and 4 feet wide. The stone fill is surrounded by filter fabric and covered with 6 inches of top soil. Scheme 5 is intended to act as a passive line of defense against future bank retreat and as a reference for monitoring bank changes. Plate 13 shows details of Scheme 5.

D. Costs The contractor received notice to proceed on 14 August 1979 and the final inspection was held on 31 January 1980. The final contract cost was $189,727, which included $29,803.35 in overruns for stone fill and pervious fill and $1,377.00 for two modifications. Unit prices included $23 per ton for graded stone, $23.50 per ton for stone fill, $17 per ton for pervious fill, $22 per ton for graded filter material, $3.50 per square yard for filter fabric, and $12 per cubic yard for unclassified excavation. Four supplemental gradation tests of the stone fill were made for record purposes at a cost of $600 each. The final construction cost per linear foot and per square foot of protection for each scheme was as follows:

SCHEME	COST PER LINEAR FOOT	COST PER SQUARE FOOT
1A	$122	$3.92
1B	$106	$3.92
2	$ 91	$3.26
3	$ 41	$3.70
4A	$ 30	*
4B	$ 86	**
5	$ 38	$5.44

* $3,000 cost per HARDPOINT

** $8,570 cost per HARDPOINT

The supervision and inspection cost was $6,300 and the engineering and design cost was $46,500.

IV. PERFORMANCE OF PROTECTION

A. Monitoring Program The Pittsburgh District Section 32 monitoring program is summarized on Plate 14. The site is instrumented with a river gage and a recording wind measuring device located at Lock No. 9. Air temperature and precipitation are also monitored at the lock. Monitoring inspections by project designers have been made at intervals averaging three months. These inspections include visual observations and photographs taken from fixed reference points. Overbank cross sections were surveyed before construction, in July 1978 and in June 1979. The site will be resectioned in 1981 and 1983. Controlled low level vertical aerial photography was taken in the spring of 1974 and in the fall of 1978. Plate 15 shows a vertical aerial photograph of the site taken in October 1980.

B. Evaluation of Protection Performance

1. General

Overall project performance has been effective through the first full year of monitoring. Ice conditions in the first winter of operation were less severe than normal and there was no overbank flooding during the ice breakup.

2. Scheme 1

Both Schemes 1A and 1B resisted the forces of ice and water with no signs of distress. There has been no apparent gouging, settlement, or loss of stone protection. Sequential photos of Scheme 1 are shown on Plates 16, 17, 18 and 19.

3. Scheme 2

The longitudinal stone dike is undamaged, however, the sand and gravel fill shows signs of gouging by ice and disturbance by high river flows. Plates 20 and 21 show photos of Scheme 2.

4. Scheme 3

The stone fill placed against the bank face has remained intact with no signs of disturbance. Photos of Scheme 3 are shown on Plates 22 and 23.

5. Scheme 4

The stone hardpoints were placed perpendicular to the movement of the ice floes and, thus, are impacted directly by the dynamic ice forces. Despite relatively mild ice conditions the riverward corners of all the hardpoints were torn away by the moving ice. Between hardpoints the bank was not significantly damaged by ice, however, later high water periods resulted in some bank loss. Plate 24 shows a typical hardpoint damaged by moving ice. One of the property owners was present during ice breakup and submitted a letter with the following graphic description of ice damage to the hardpoints:

"Mrs. Steiner and I were standing on the river bank on the Brumbaugh property when the ice started to move a few weeks ago. The ice struck the hardpoints with such force that the stone was immediately dislodged, some, of it was thrown up into the air, some pushed into the water and others thrown on to ice floes and carried downstream."

Although significantly damaged by ice, the hardpoints will not be repaired or replaced immediately. Reconstruction plans will be deferred until further experience is gained. Plates 25, 26, 27, 28, 29, 30,, 31 and 32 show sequential photos of Scheme 4.

6. Scheme 5

Although minor bank recession has occurred since construction, none of the stone fill has been exposed. Plate 33 shows photos of the riverbank along Scheme 5.

C. Rehabilitation. No rehabilitation has been performed. An

eventual rehabilitation of Scheme 4 is anticipated, however, the nature and magnitude will depend upon future performance.

D. <u>Summary of Findings.</u> Stone revetments of various designs appear to resist ice attack and other erosive forces successfully, as demonstrated by the performance of Schemes 1 and 3. The gently sloping sand and gravel fill of Scheme 2 has effectively absorbed the forces of moving ice and protected the bank face from erosion by high water. The stone hardpoints of Scheme 4, while protecting the bank from moving ice, were significantly damaged. Further observations will show whether the hardpoints will stabilize or continue to deteriorate.

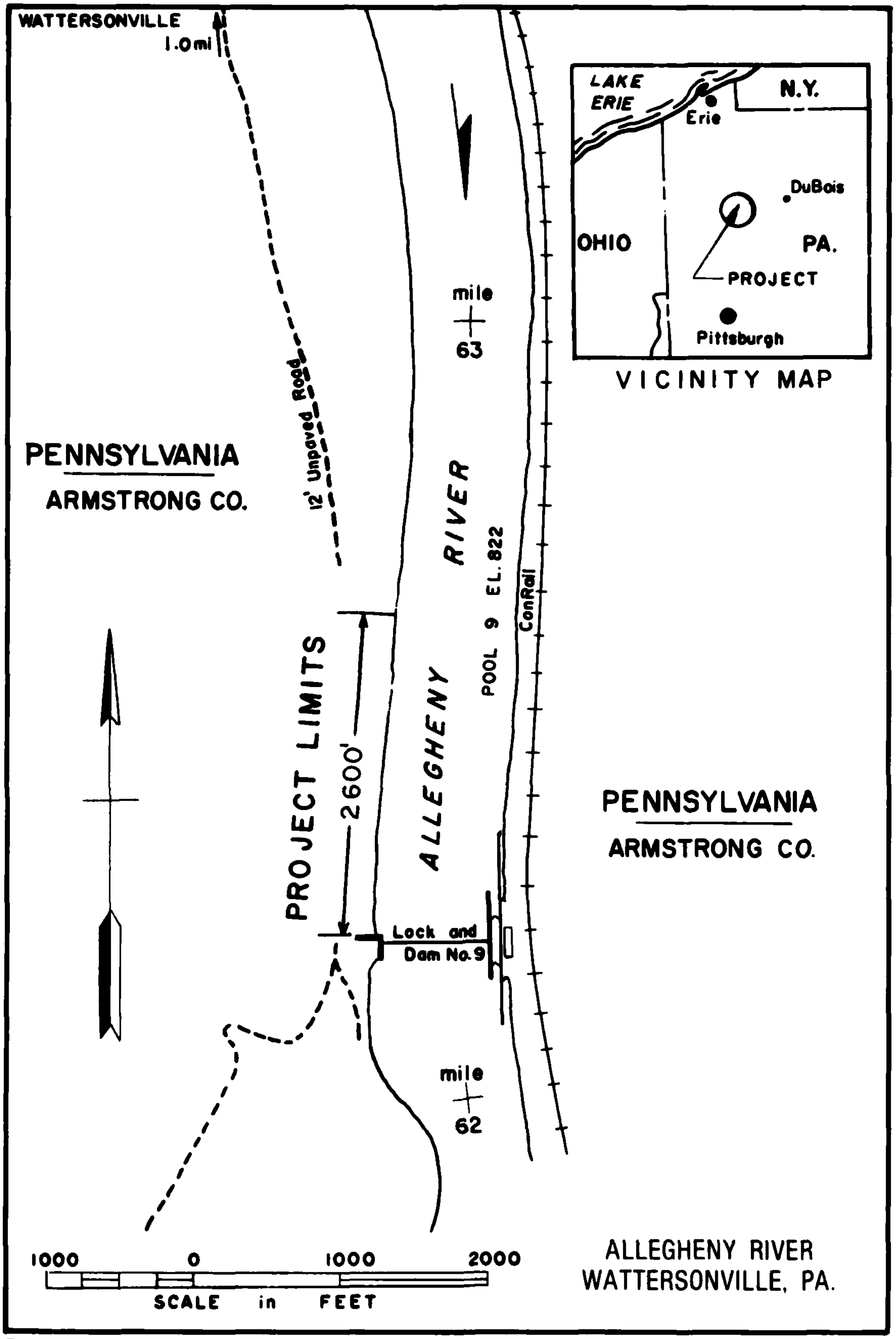
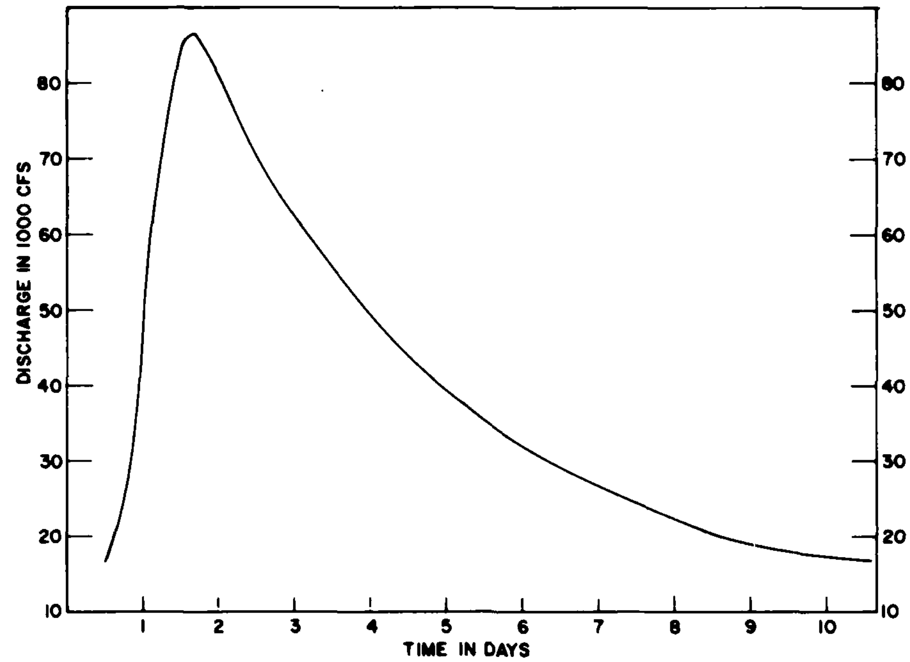
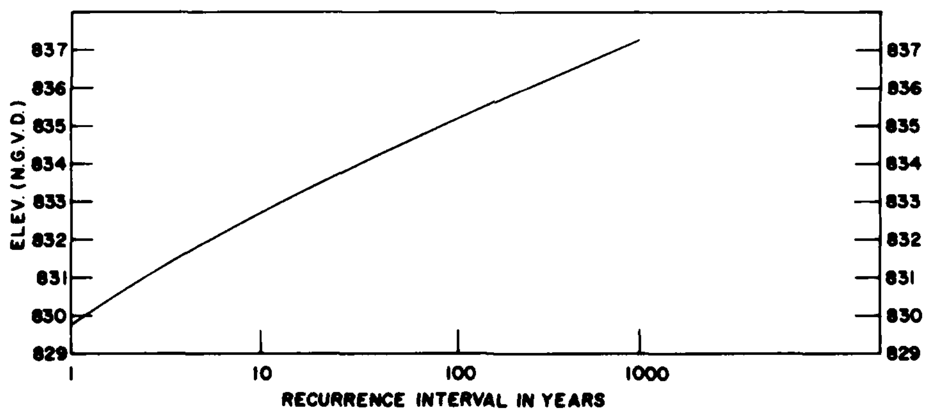

WATTERSONVILLE
1.0 mi
12' Unpaved Road
PENNSYLVANIA
ARMSTRONG CO.
mile
63
ALLEGHENY RIVER
POOL 9 EL. 822
ConRail
PROJECT LIMITS
2600'
Lock and Dam No. 9
mile
62
LAKE ERIE
Erie
N.Y.
DuBois
OHIO
PA.
PROJECT
Pittsburgh
VICINITY MAP
PENNSYLVANIA
ARMSTRONG CO.
1000
0
1000
2000
SCALE in FEET
ALLEGHENY RIVER
WATTERSONVILLE, PA.

PLATE 1



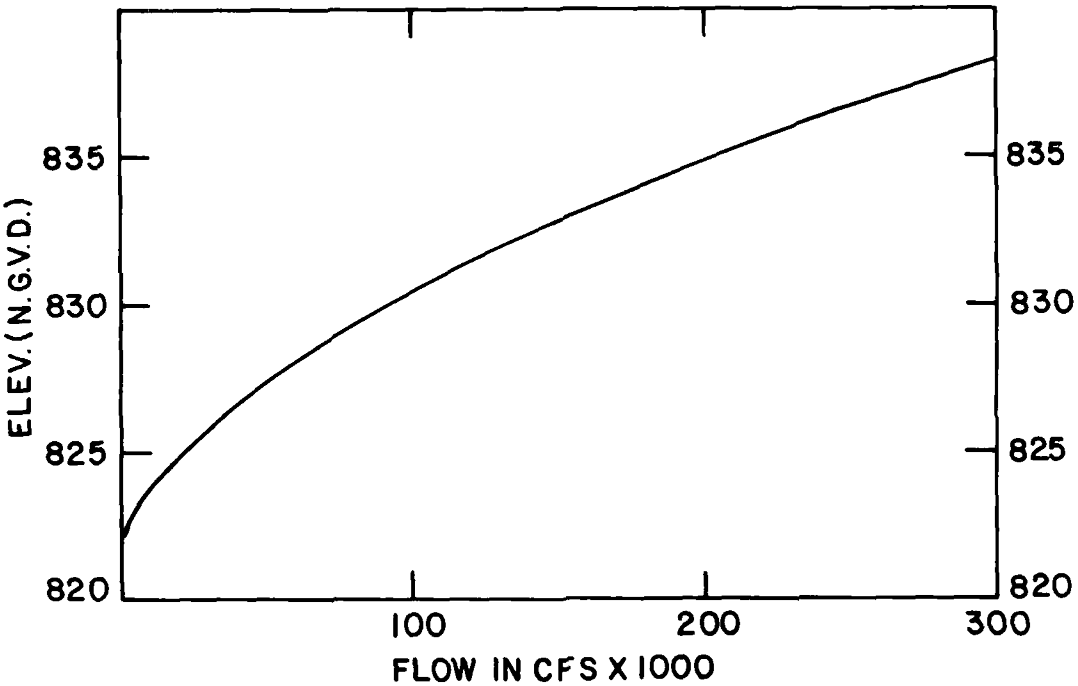
ONE YEAR FLOOD HYDROGRAPH



STAGE FREQUENCY CURVE

ALLEGHENY RIVER
WATTERSONVILLE, PA.

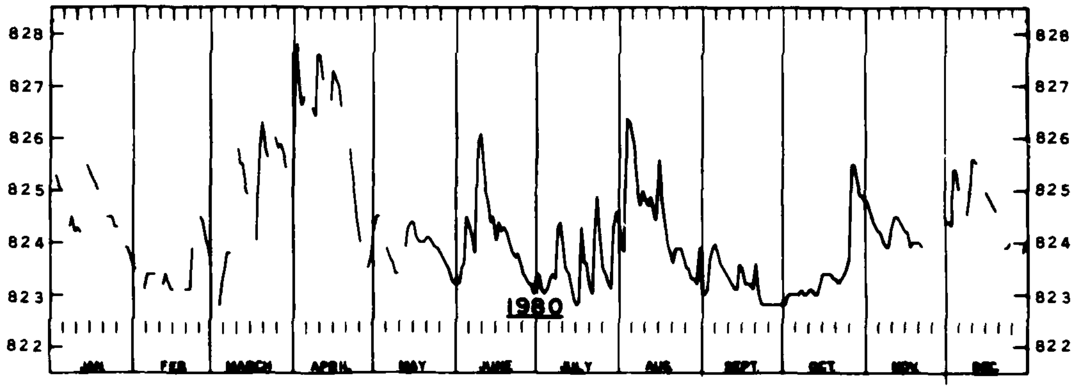
PLATE 2



DISCHARGE RATING CURVE

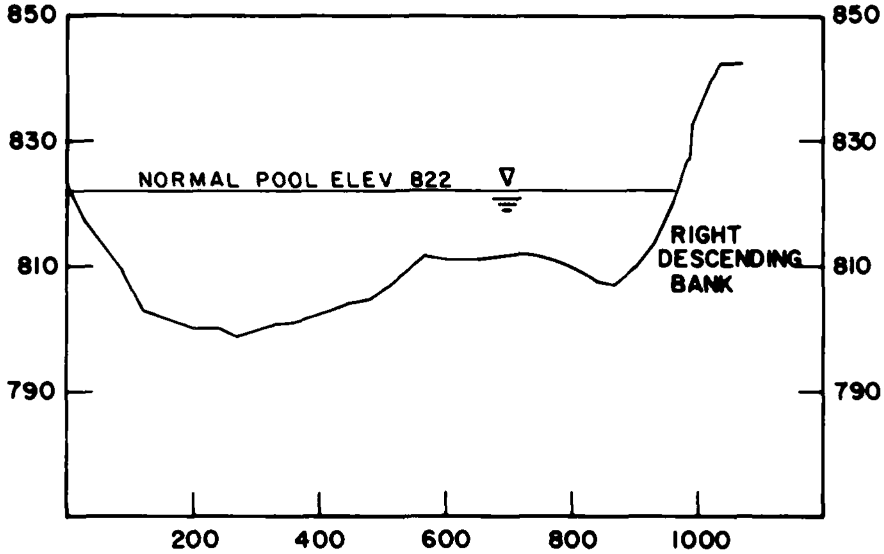
ALLEGHENY RIVER
WATTERSONVILLE, PA.
DISCHARGE RATING CURVE

PLATE 3

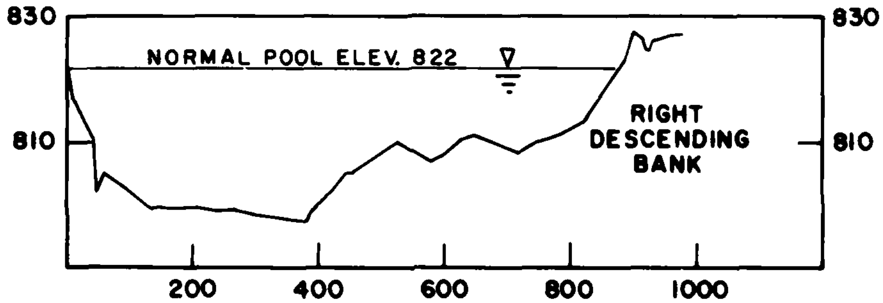


ALLEGHENY RIVER
WATTERSONVILLE, PA.
HYDROGRAPH

PLATE 4



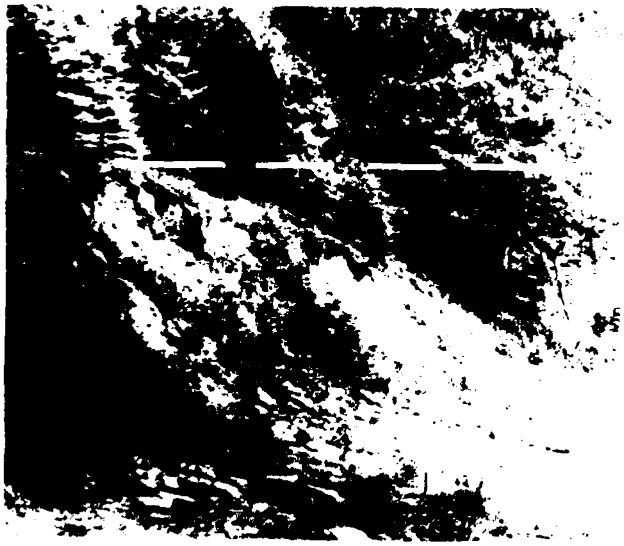
STATION 21+00



STATION 34+00

ALLEGHENY RIVER
WATTERSONVILLE, PA.
CHANNEL CROSS SECTIONS

PLATE 5



SILTY SAND WITH INTER-LENSING SANDY SILT AND CLAYEY SILT (broken rock debris on beach from exposed old foundation)

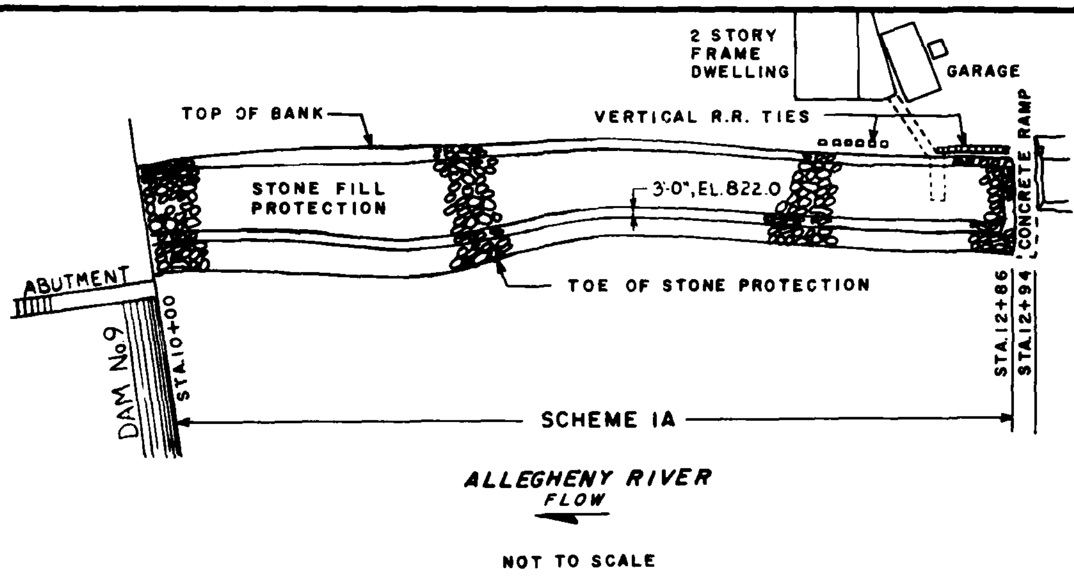
UPSTREAM VIEW FROM
STATION 17+00
19 JULY 1978

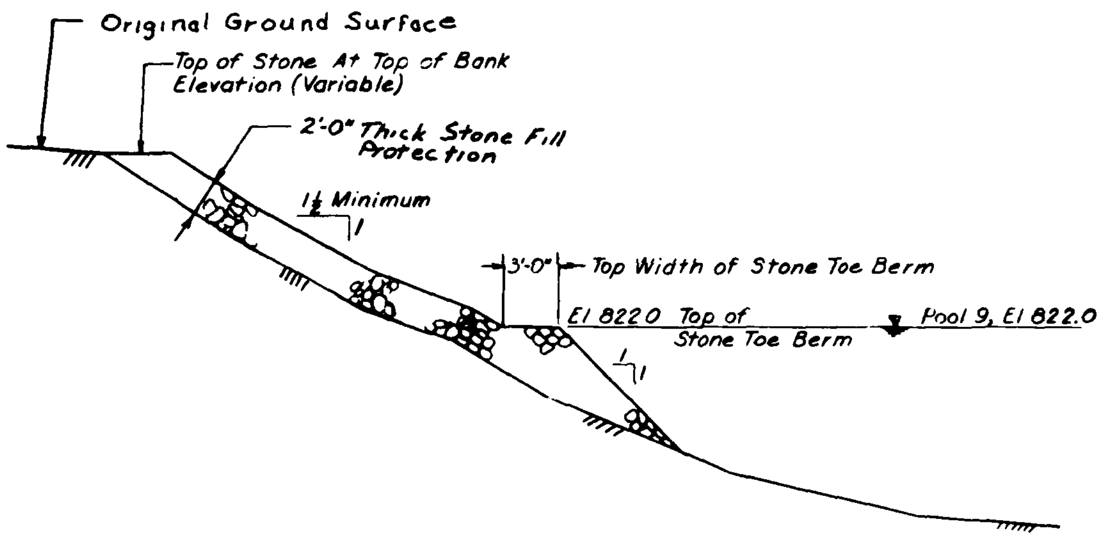
THINLY INTERBEDDED CLAYEY SILT AND SANDY SILT WITH SOME SANDSTONE ARTIFACTS

DOWNSTREAM VIEW FROM
STATION 28+00
19 JULY 1978

ALLEGHENY RIVER
WATTERSONVILLE, PA.
RIVERBANK SOILS

PLATE 6



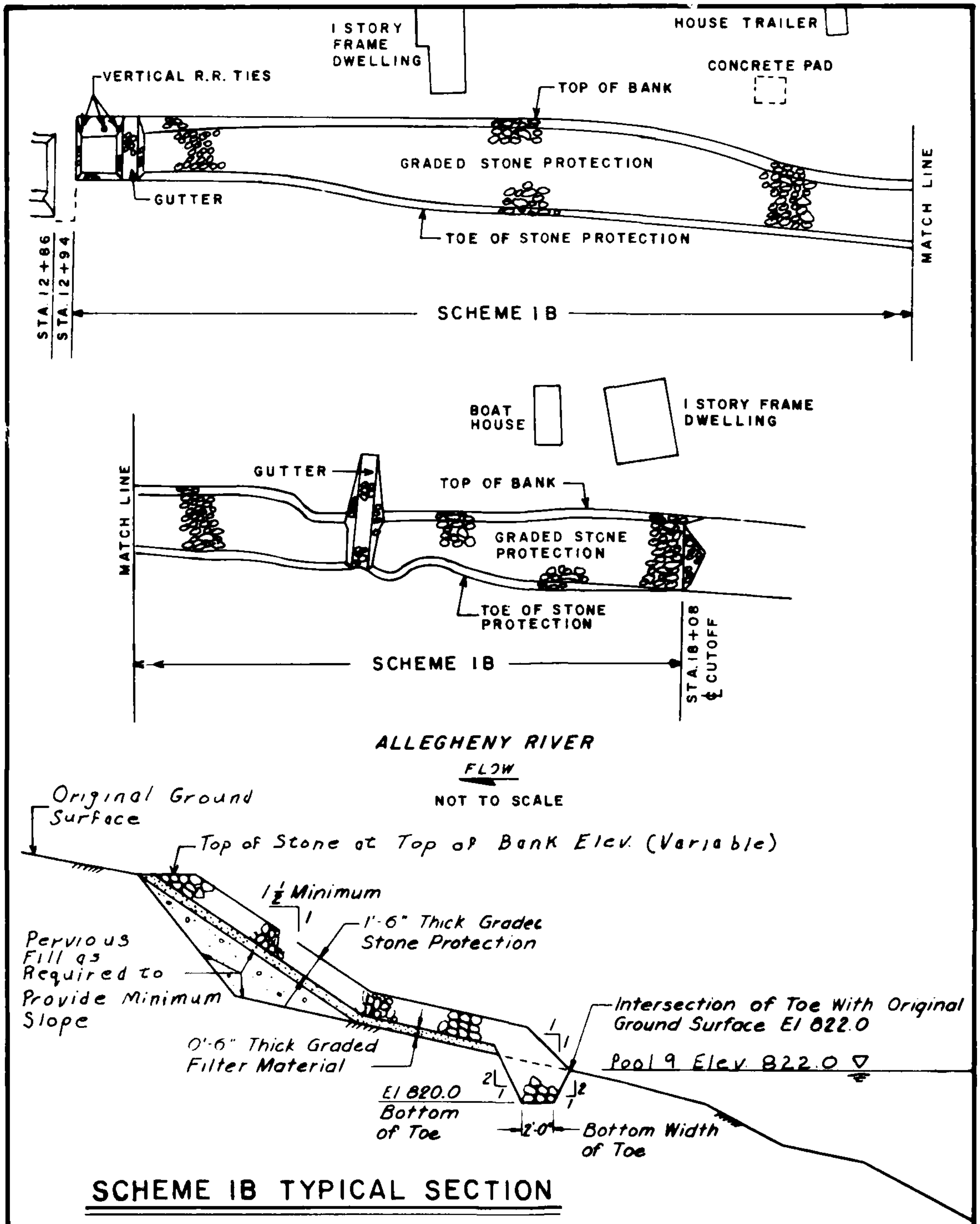


SCHEME 1A TYPICAL SECTION

STA 10+00 TO STA. 12+86
NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1A

PLATE 7

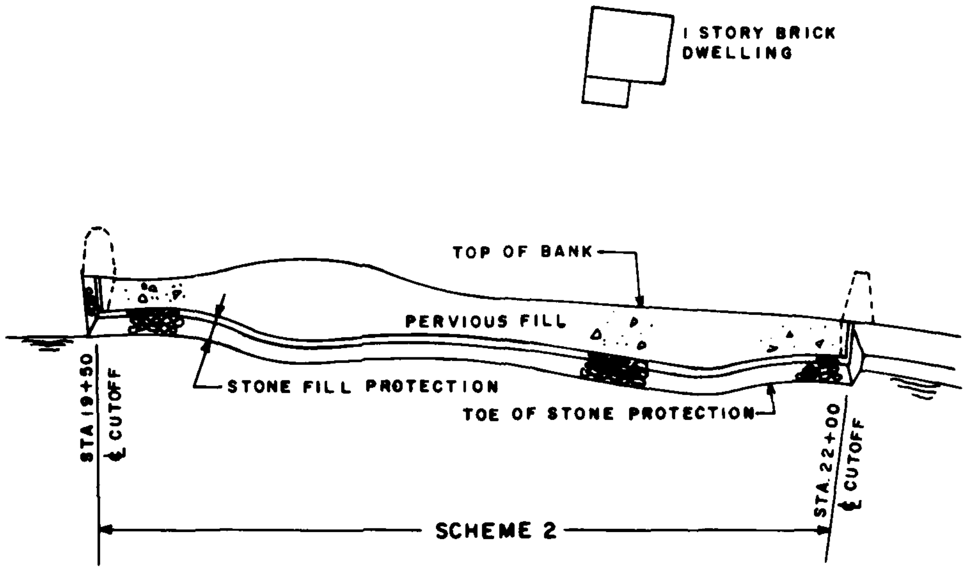


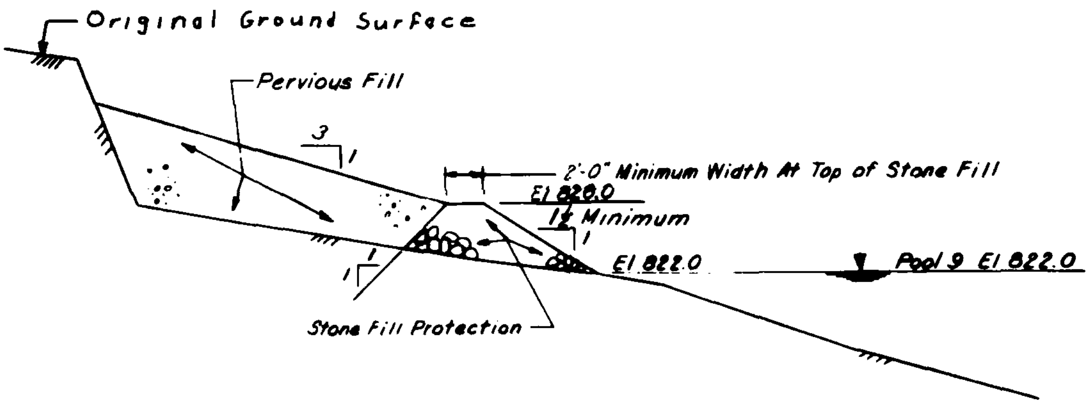
SCHEME 1B TYPICAL SECTION

STA 12+94 TO STA.18+08
NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1B

PLATE 8



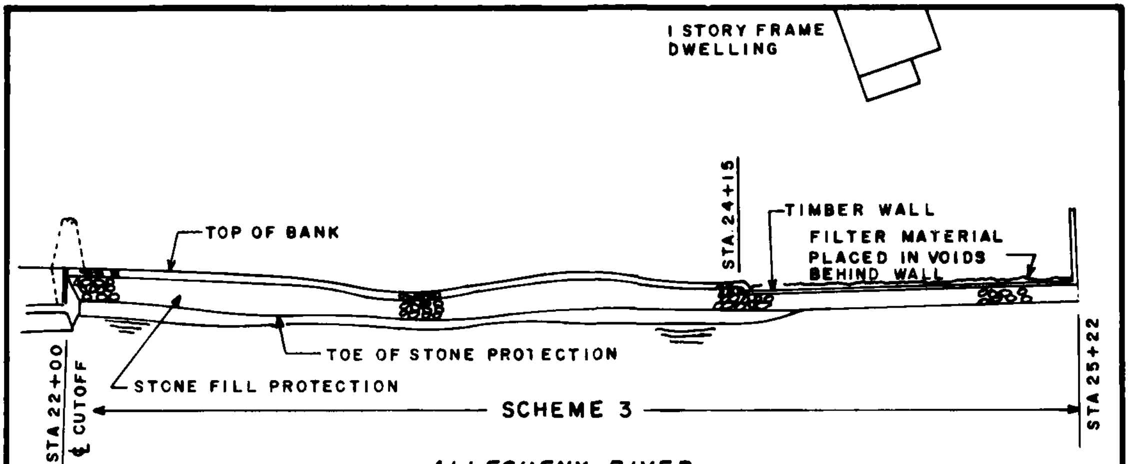


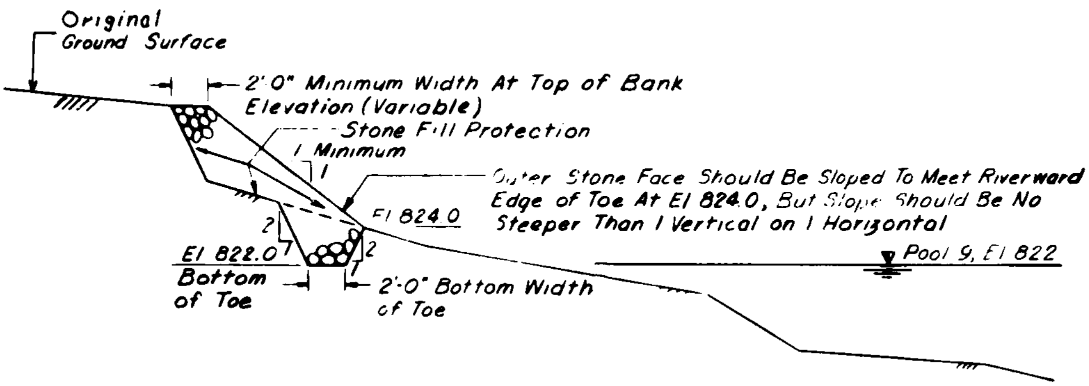
SCHEME 2 TYPICAL SECTION

STA 19+50 TO STA. 22+00
NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 2

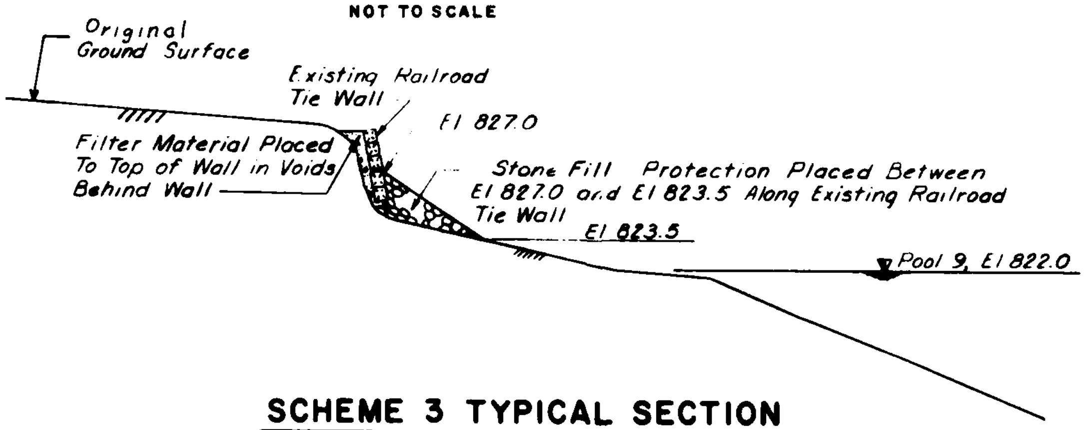
PLATE 9





SCHEME 3 TYPICAL SECTION

STA 22+00 TO STA 24+15
NOT TO SCALE

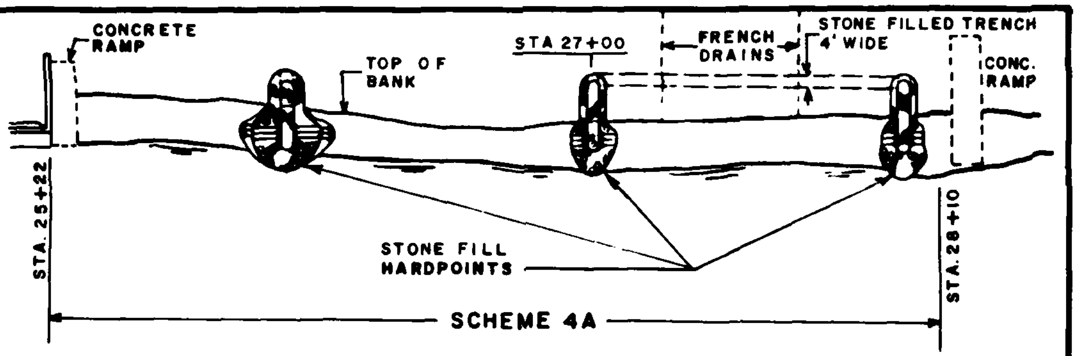


SCHEME 3 TYPICAL SECTION

(AT EXISTING RAILROAD TIE WALL)
STA 24+15 TO STA 25+22
NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 3

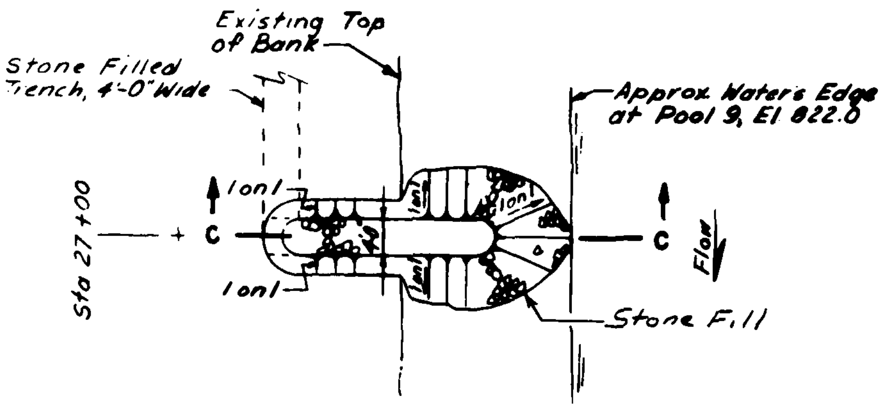
PLATE 10



ALLEGHENY RIVER

FLOW

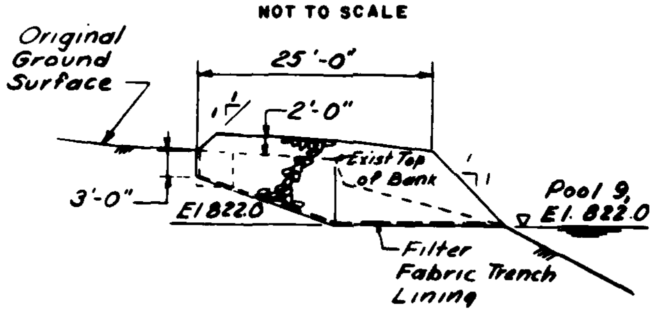
NOT TO SCALE



PLAN

STA 27+00

NOT TO SCALE



SECTION C–C

NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4A

PLATE 11

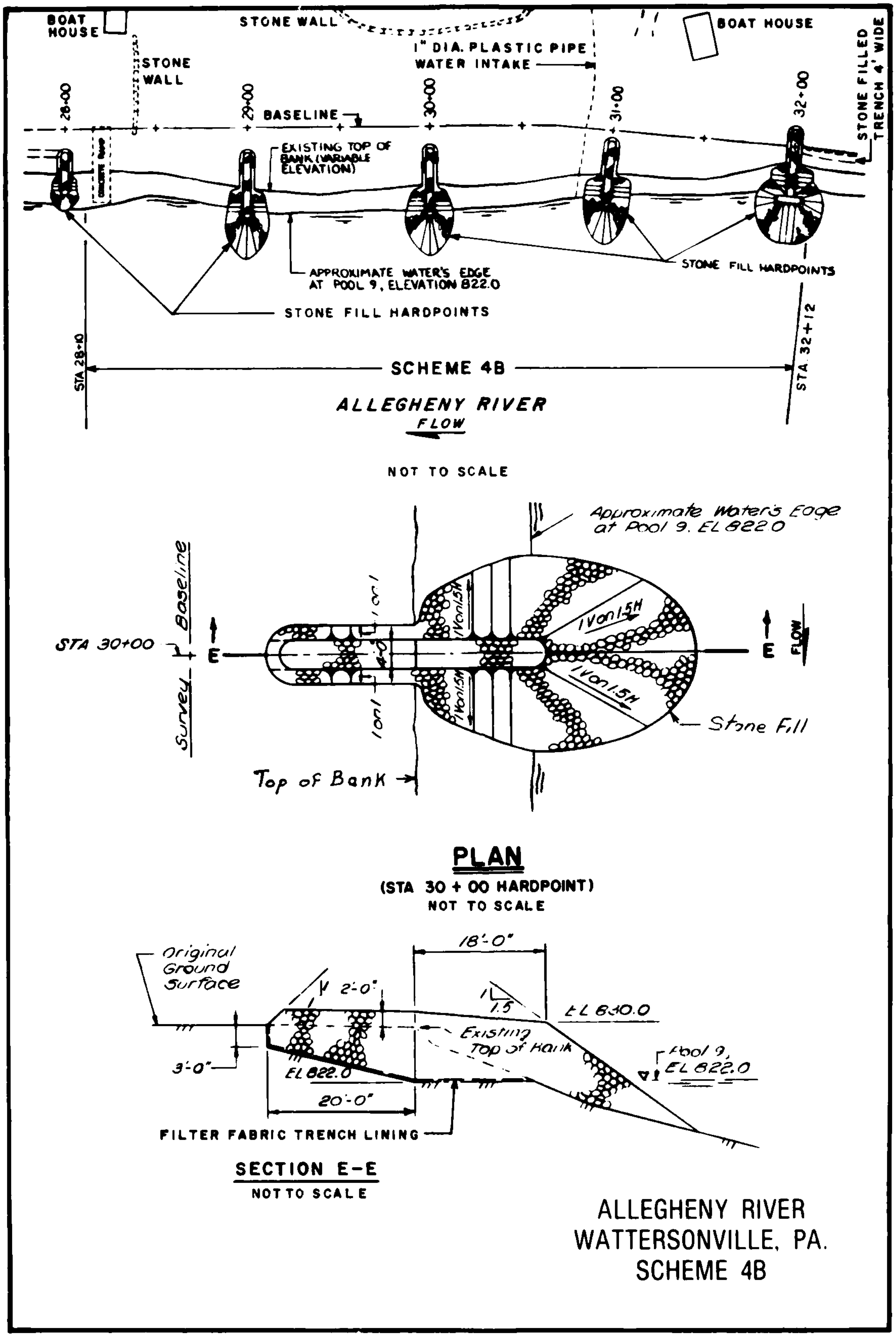
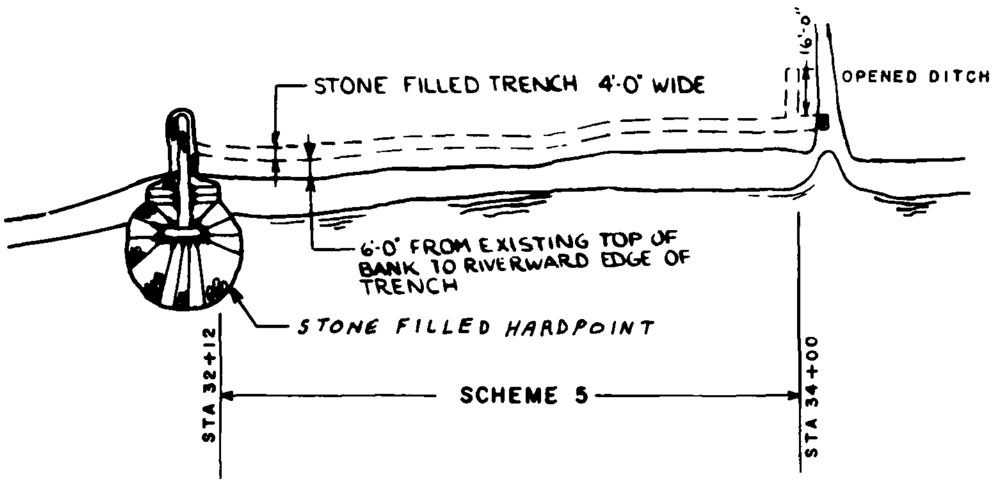

BOAT HOUSE
STONE WALL
BOAT HOUSE
STONE WALL
1" DIA. PLASTIC PIPE
WATER INTAKE
STONE FILLED TRENCH 4' WIDE
28+00
29+00
30+00
31+00
32+00
BASELINE
EXISTING TOP OF BANK (VARIABLE ELEVATION)
APPROXIMATE WATER'S EDGE AT POOL 9, ELEVATION 822.0
STONE FILL HARDPOINTS
STONE FILL HARDPOINTS
STA 28+10
STA 32+12
SCHEME 4B
ALLEGHENY RIVER
FLOW
NOT TO SCALE
Approximate Water's Edge at Pool 9. EL 822.0
Survey Baseline
STA 30+00
E
E
FLOW
1V on 1.5H
1V on 1.5H
1V on 1.5H
1V on 1.5H
1 on 1
1 on 1
Stone Fill
Top of Bank
PLAN
(STA 30 + 00 HARDPOINT)
NOT TO SCALE
Original Ground Surface
18'-0"
2'-0"
1
1.5
EL 830.0
Existing Top of Bank
3'-0"
EL 822.0
Pool 9, EL 822.0
20'-0"
FILTER FABRIC TRENCH LINING
SECTION E-E
NOT TO SCALE
ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B

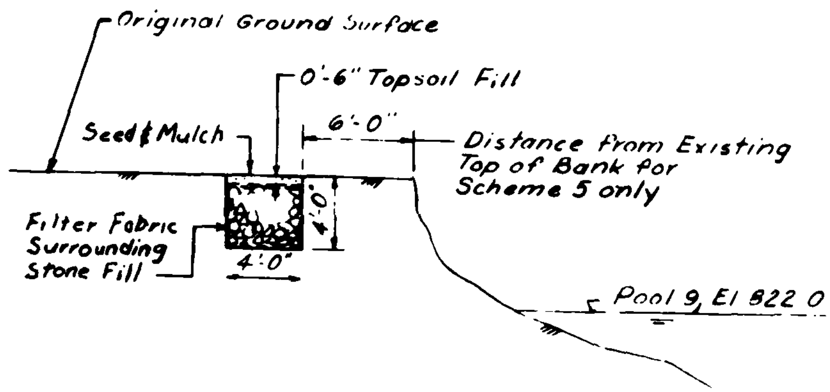
PLATE 12



ALLEGHENY RIVER

FLOW

NOT TO SCALE



SCHEME 5 TYPICAL SECTION

STA.32+12 TO STA.34+00

NOT TO SCALE

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 5

PLATE 13

PARAMETER	ITEM	FREQUENCY
GEOMETRY	1. Cverbank crossection from baseline to 50 ft. riverward of water's edge at 50 ft. intervals	Biyearly
	2. Full channel crossections	Once
	3. Ground photos from fixed reference points	Monthly
	4. Controlled vertical low level aerial photos	Annual
CLIMATE	1. Air temp., precip., wind direction and velocity (weather station)	Continuous
	2. Ice conditions, snow cover noted from visual observations	As available
HYDRAULICS	1. River stage record from staff gage at Lock & Dam 9	Twice daily
	2. Wave action visual observation	As available
	3. River traffic (through observation and lock records)	As available
STREAM-BANK PROTECTION	1. Monitor dimensional changes of marked structural & vegetal units through photos and manual measurement	Monthly
	2. Observe durability of marked units of structural material (qualitative)	Monthly
	3. Observe condition of marked plants	Monthly
	4. Record initiation and measure progression of failures in bank protection	Monthly
GEOLOGY AND SOILS	1. Material properties testing	Annual

ALLEGHENY RIVER
WATTERSONVILLE, PA.
MONITORING PROGRAM

PLATE 14

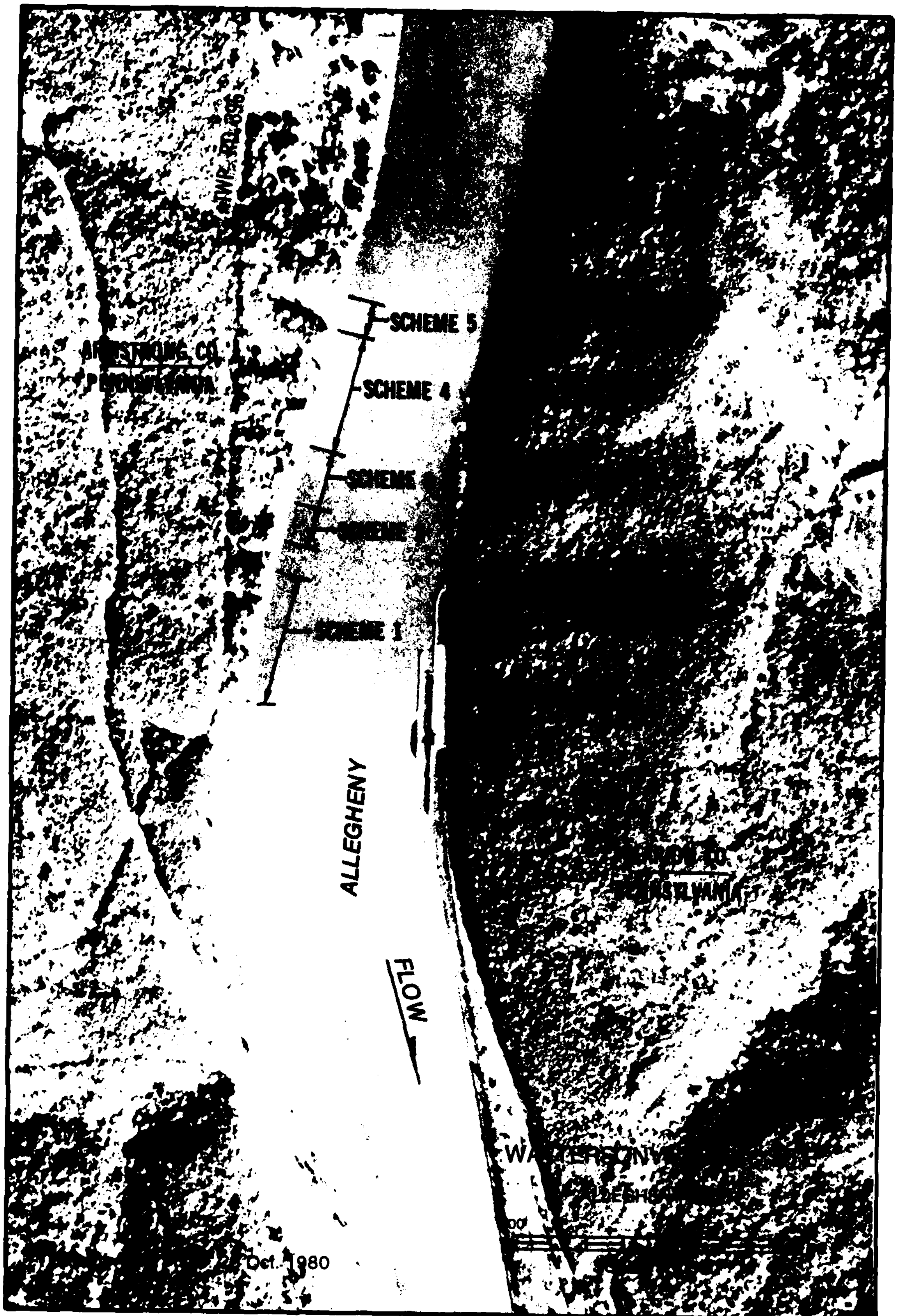


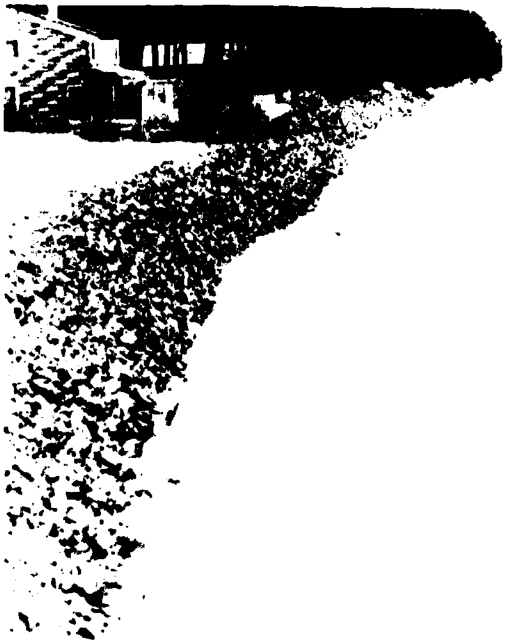
PLATE 15

BEFORE CONSTRUCTION
FROM STATION 10+00
12 DECEMBER 1978

DURING CONSTRUCTION FROM
STATION 10+00
20 DECEMBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1A PHOTOS

PLATE 16



RIVER FROZEN OVER, FROM
STATION 10+00
12 FEBRUARY 1980

FROM STATION 10+00
27 JUNE 1980

STACKED ICE ALONG SHORE
FROM STATION 10+00
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1A PHOTOS

PLATE 17

BEFORE CONSTRUCTION
LOOKING DOWNSTREAM FROM STATION 15+50
19 JULY 1978

LOOKING DOWNSTREAM
FROM STATION 15+50
20 DECEMBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1B PHOTOS

PLATE 18

RIVER FROZEN OVER, LOOKING DOWNSTREAM FROM STATION 15+50
12 FEBRUARY 1980

FROM STATION 15+50
27 JUNE 1980

STACKED ICE ALONG SHORE
FROM STATION 15+50
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 1B PHOTOS

PLATE 19

BEFORE CONSTRUCTION
FROM STATION 19+50
1 MAY 1979

DURING CONSTRUCTION
FROM STATION 19+50
15 NOVEMBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 2 PHOTOS

PLATE 20

RIVER FROZEN OVER
FROM STATION 19+50
12 FEBRUARY 1980

FROM STATION 19+50
9 APRIL 1980

STACKED ICE ALONG SHORE FROM
STATION 19+50
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 2 PHOTOS

PLATE 21

BEFORE CONSTRUCTION
FROM STATION 24+00
1 MAY 1979

DURING CONSTRUCTION
FROM STATION 23+20
15 NOVEMBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 3 PHOTOS

PLATE 22

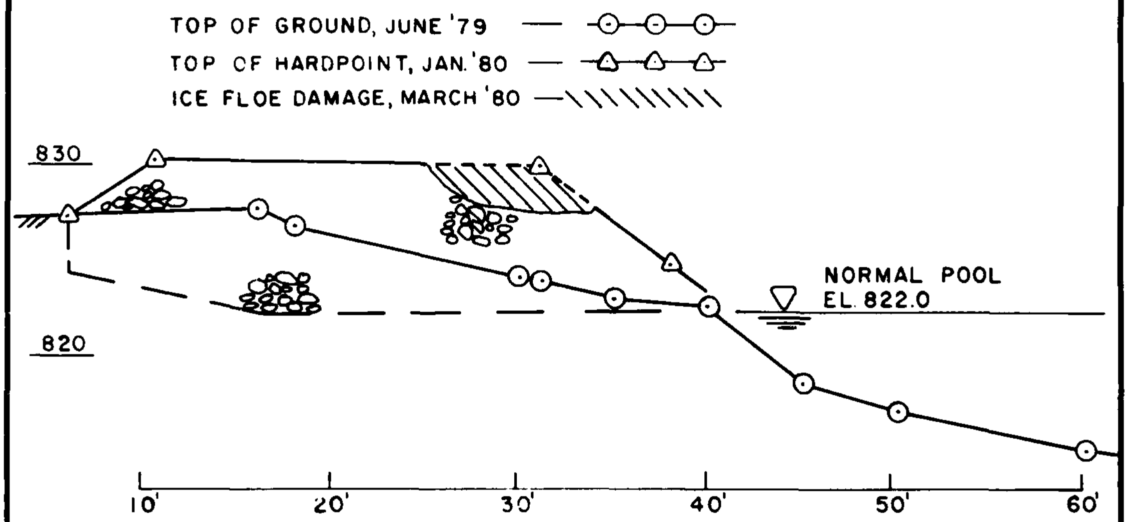
RIVER FROZEN OVER
FROM STATION 24+00
12 FEBRUARY 1980

FROM STATION 24+00
27 JUNE 1980

STACKED ICE ALONG SHORE
FROM STATION 24+00
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 3 PHOTOS

PLATE 23



STATION 26+00 HARDPOINT

STATION 26+00 HARDPOINT
27 JUNE 1980

ALLEGHENY RIVER
WATTERSONVILLE, PA.
DAMAGED HARDPOINT

PLATE 24

BEFORE CONSTRUCTION
STACKED ICE ALONG RIVER BANK
TAKEN FROM STATION 28+00
28 FEBRUARY 1977

BEFORE CONSTRUCTION
LOOKING UPSTREAM FROM STATION 27+00
1 MAY 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4A PHOTOS

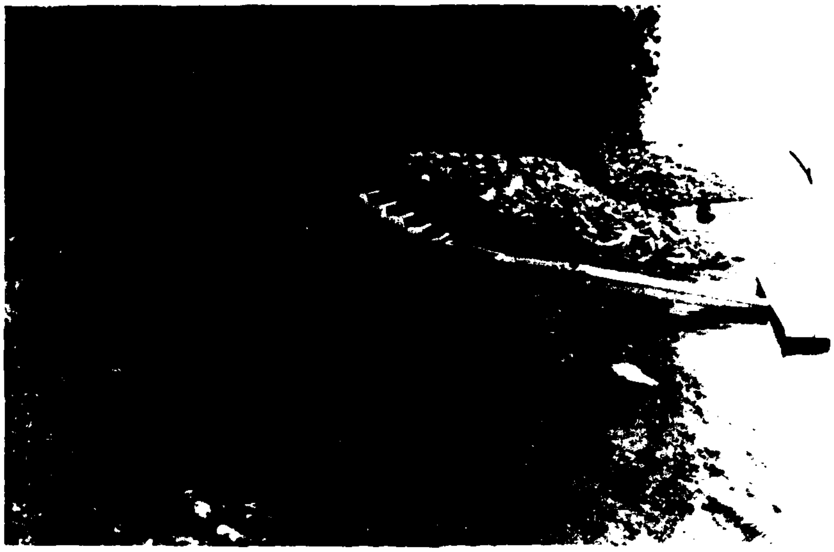
PLATE 25

RIVER FROZEN OVER
TAKEN FROM STATION 27+00
12 FEBRUARY 1980

FROM STATION 27+00
NOTE DAMAGED NOSE OF HARDPOINT
9 APRIL 1980

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4A PHOTOS

PLATE 26

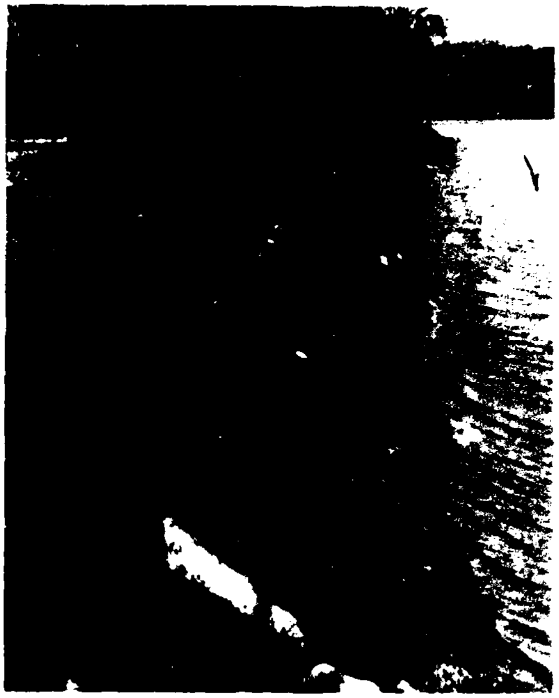


FROM STATION 27+00
27 JUNE 1980

STACKED ICE ALONG RIVERBANK
FROM STATION 27+00
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4A PHOTOS

PLATE 27



BEFORE CONSTRUCTION
LOOKING UPSTREAM
FROM STATION 30+50
1 MAY 1979

DURING CONSTRUCTION
VIEW OF HARDPOINT AT STATION 32+00
FROM STATION 31+00
30 OCTOBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B PHOTOS

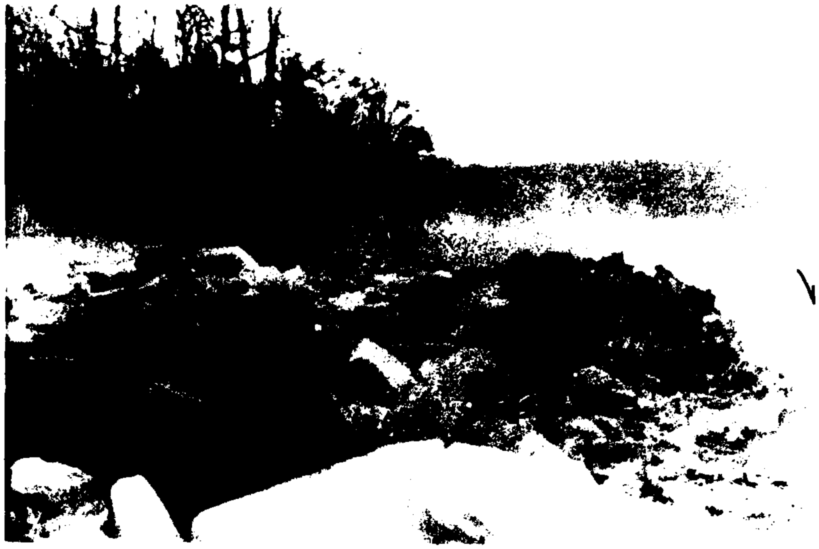
PLATE 28

RIVER FROZEN OVER
VIEW FROM STATION 31+00
NOTE SHEARED ICE ZONE BEYOND NOSE OF HARDPOINT
12 FEBRUARY 1980

VIEW FROM STATION 31+00
NOTE ICE FLOW DAMAGE TO NOSE OF HARDPOINT
27 JUNE 1980

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B PHOTOS

PLATE 29



VIEW FROM STATION 31+00 OF STACKED
ICE OVER HARDPOINT AT STATION 32+00
20 FEBRUARY 1981

VIEW FROM STATION 31+00
NOTE ICE DAMAGE TO NOSE OF HARDPOINT
14 APRIL 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B PHOTOS

PLATE 30

ICE FLOE GOUGING OF RIVER BANK
AT STATION 30+50
28 FEBRUARY 1977

VIEW OF SCHEME 4 FROM HARDPOINT
AT STATION 32+00
10 DECEMBER 1979

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B PHOTOS

PLATE 31

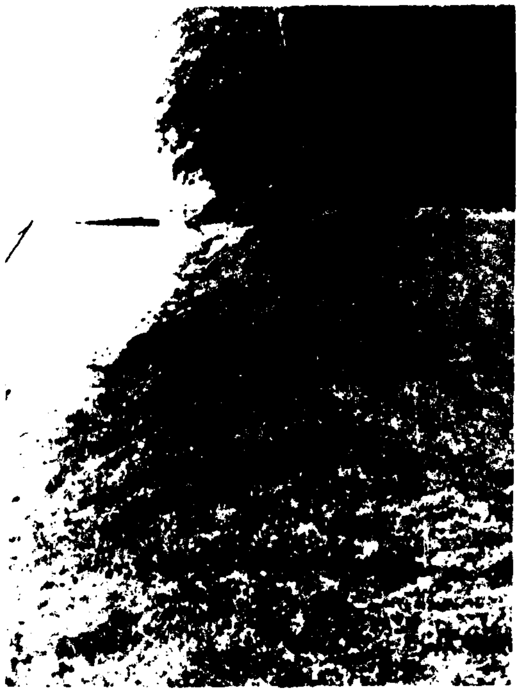
VIEW OF HARDPOINTS FROM HARD-
POINT AT STATION 32+00, NOTE
SHEARED ICE ZONE BEYOND NOSES
OF HARDPOINTS
12 FEBRUARY 1980

VIEW LOOKING
DOWNSTREAM OF
ICE OVERRIDING
HARDPOINT AT
STATION 30+00
20 FEBRUARY 1981

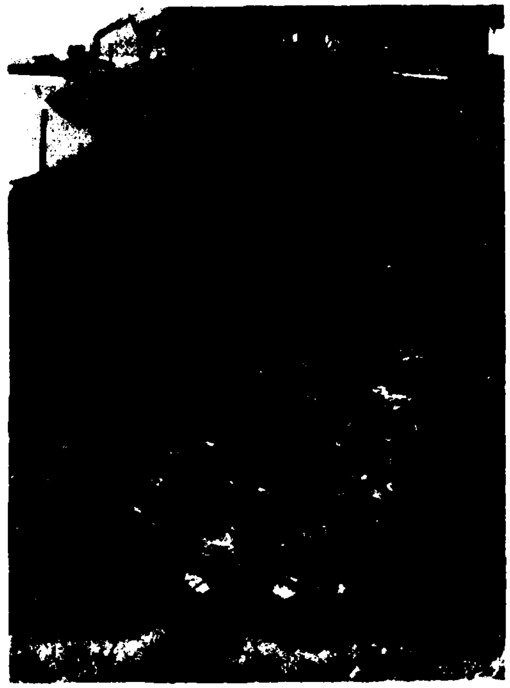
VIEW OF HARD-
POINT AT STATION
30+00, NOTE ICE
STACKED UPSTREAM
OF HARDPOINT AND
ABSENCE OF ICE
IMMEDIATELY DOWN-
STREAM OF HARD-
POINT
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 4B PHOTOS

PLATE 32



BEFORE CONSTRUCTION
VIEW FROM STATION 34+00
19 JULY 1978



DURING CONSTRUCTION
VIEW FROM STATION 34+25
2 NOVEMBER 1979

VIEW FROM STATION 34+50
ICE STACKED ALONG THE SHORE
20 FEBRUARY 1981

ALLEGHENY RIVER
WATTERSONVILLE, PA.
SCHEME 5 PHOTOS

PLATE 33

**CONNECTICUT RIVER AT
HAVERHILL, NEW HAMPSHIRE**

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

CONNECTICUT RIVER AT HAVERHILL, NEW HAMPSHIRE
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Haverhill Demonstration Site, Connecticut River, Haverhill, New Hampshire. Location map is shown on Plate 1.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a demonstration project constructed by the New England Division to experiment with innovative techniques of streambank erosion control. It presents a description of the bank erosion problem and types of protection used and provides an evaluation of performance experience to date.

4. Problem Résumé. The left bank of the Connecticut River in the project area is 7 to 22 feet above the normal water level and is eroding at an average rate of 10 feet per year. Annual spring high water inundates the lower bank and the whole bank is inundated by unusually high spring runoff or floodflows. Erosion conditions at the site are typical of those caused by shear and eddy forces acting on alluvial soils on the outside of a bend in a meandering river. Ice also plays a role in the overall erosion process as do other less significant factors. The land being lost is prime farmland and there is likelihood that continued erosion would create an oxbow cutoff resulting in the isolation and loss of 30 acres of farmland.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The project site lies in the northern portion of the Connecticut River Basin, the largest in New England. Draining

in a southerly direction with its source in northern New Hampshire and mouth at Long Island Sound in Saybrook Connecticut, the Connecticut River extends more than 400 miles. The valley of the Connecticut River in the project area (approx. river mile 255) is bounded on the west by the Green Mountains of Vermont and on the east by the White Mountains of New Hampshire, with mountain peak elevations of over 4000 feet NGVD being quite common. Made up mostly of a glacial outwash plain, the valley floor is relatively narrow and of gentle terrain with elevations in the project area being about 400 feet NGVD. The normal stream gradient at the Haverhill site is about 0.1 foot per mile.

b. Geology. The geology of the Connecticut River Basin can be subdivided into two distinct periods: prior to and following continental glaciation. Preglacial history of the Connecticut River is quite diverse. Bedrock of the area consists of heavily folded and faulted metamorphic and igneous rocks. The metamorphic rocks include phyllites, schists, and gneisses. The igneous bodies are granite, granodiorite, and quartz monzanite with occasional intrusions of volcanic materials. The trends of major structural features in Vermont and New Hampshire are in a north-northeasterly direction. This coincides with the Connecticut River which probably follows an ancient drainage way.

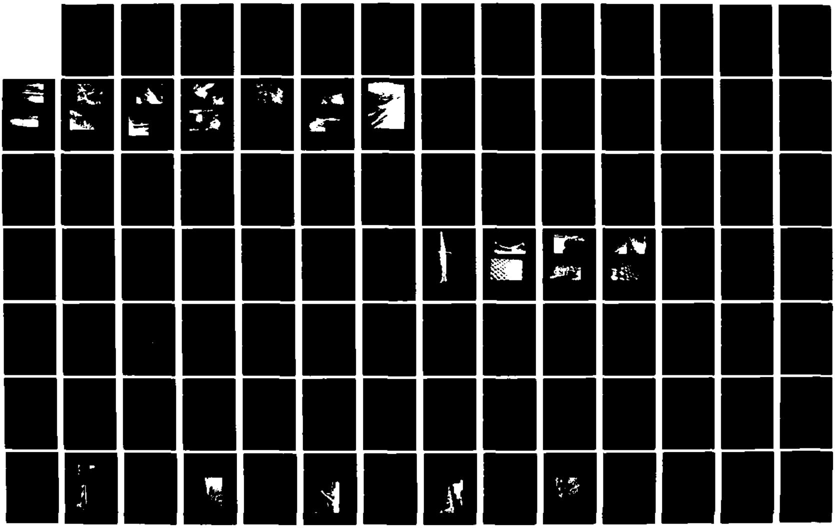
Preglacial geology indicates extensive periods of erosion associated with the uplift of the Appalachian Mountains near the close of the Paleozoic Era, and with other periods when the land was emergent. It is assumed that the present topography was well established prior to continental glaciation, including a well developed soil layer with superimposed streams including their meandering patterns.

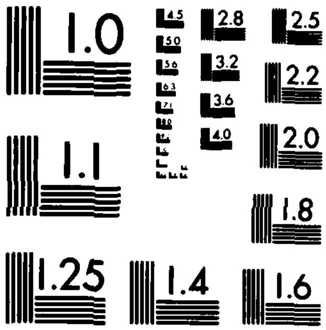
Massive continental glaciation wore the topography into the currently existing subdued forms. Highlands were rounded on the upper side facing the glacier and steepened on the lower side away from the glacier. Stream valleys were eroded and smoothed, sometimes into the classic V-shaped glacial valleys.

The retreating ice redeposited morainal materials over the entire

AD-A121 138 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. 3/4

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

surface of the area. Stagnant ice blocks and frontal moraines created lakes that became sites of further deposition. While the lakes were in existence, material was deposited as sandy and gravelly terraces consisting of deep, well drained alluvial soils developing in medium textured sediments, derived mainly from schist, gneiss, granite, slate and phyllite. The type of alluvial material forming the flood plains adjacent to the Connecticut River comprise the Hadley soils as classified by the US Department of Agriculture, Soil Conservation Service, and are water-laid deposits composed mostly of silty, fine sands and nonplastic fine, sandy silts. Soil core samples were collected and analyzed. The soil profile of the alluvial deposits is given on Plate 2.

c. Locality, Development and Occupation. The Haverhill, New Hampshire site is located in the upper Connecticut River valley, an area of rich alluvial soil which is the basis for one of the most productive farming areas in New England. Crops produced in the region include corn, hay and other assorted feed crops. The erosion site borders a large open field used primarily for corn.

Vegetation in the river valley is limited to small areas not in agriculture, including some woodlots and woodland along the river in areas not subject to streambank erosion. Woodland types include red and silver maple, elm, willow, cherry, poplar and alders. Some timber harvesting is done for lumber and firewood.

The population of Haverhill (3000 in 1970) and vicinity has declined in the last decade. Most of the labor force consists of people engaged in manufacturing, light industry, agriculture, and the tourist trade. There is also a significant amount of self-employed people in the region. The area at one time served as a distribution center for dairy farms in northern New Hampshire and Vermont.

d. Hydrologic Characteristics.

(1) Climatology. The relatively high elevations of the Green Mountains of Vermont and the White Mountains of New Hampshire have a marked influence on the temperature, precipitation and snow

cover of the Connecticut River Basin which lies in the path of the prevailing westerlies and air masses moving predominantly from the interior of North America. Generally west to southwest air flow brings the hot dry weather which is responsible for occasional summer droughts. In the winter months, high pressure weather systems from Canada bring frigid air into the basin. Precipitation is moderate to heavy and well distributed throughout the year. The annual mean temperature is about 45°F.

The average annual precipitation ranges from 36 inches in the main river valley to over 60 inches in the higher White and Green Mountains. Precipitation in the central and northern portions of the basin during the winter months is practically all in the form of snow. The average snowfall is from 50 to 70 inches in the valley to well over 100 inches in the mountains.

Three general types of storms produce precipitation over the basin: continental, coastal, and thunderstorms. Continental storms originate over the western and central portion of the United States and move generally in an easterly or northeasterly direction.

Tropical hurricanes, the most severe of the coastal storms originate in the South Atlantic or Caribbean Sea. They usually move in a westerly direction then northerly and may be deflected by high pressure zones to New England. Hurricanes have occurred in the summer and fall months. Extratropical storms generally originate or intensify near the middle Atlantic States, travel northward along the coastline and generally occur in autumn, winter and spring.

The third type of storm is the thunderstorm which can be produced by local convective activity during the warm humid days of the summer months or be associated with a frontal system moving across the basin.

The areal distribution of annual runoff follows a pattern somewhat similar to the annual precipitation in that it varies from 17 inches in the lower elevations to more than 40 inches in the White and Green Mountains. About 50 percent of the annual runoff occurs in the spring months of March, April and May. During this period combined rainfall runoff and snowmelt create an especially great chance of flooding.

(2) Streamflow. Flow conditions at the Haverhill site are best represented by the records of the US Geological Survey gage located 11 miles upstream on the Connecticut River at Wells River, Vermont (D.A. = 2644 square miles). The average flow at the gage during the summertime low flow season (July-October) is 3450 cfs and during the spring snowmelt months (April-May) is 15,635 cfs. The average annual peak discharge has been 33,100 cfs since completion of the last major upstream storage project in 1961. During the period from 1918-1950 the USGS recorded discharges in the Connecticut River at its gaging station in South Newbury, Vermont (D.A. = 2825 square miles) two miles downstream from the Haverhill site. Average discharge for the period of record was about 5000 cfs and a maximum discharge of 77,800 cfs was recorded on 19 and 20 March 1936. The recurrence interval of this peak discharge is estimated by statistical analysis to be about 90 years. By comparison, a minimum discharge of 198 cfs was recorded on 4 September 1934.

e. Channel Conditions. Under normal to moderate flow conditions the Connecticut River passes through the well defined channel it has cut through the valley floor alluvium (Plate 1 and Photo 1). The river takes a meandering course and, typically, is continually eroding its banks which are generally steep and caving and consist mainly of silty sands and sandy silts. Overtopping of the riverbank can occur whenever flows are in the order of 40,000 cfs, or on the average about every 3 to 5 years. At this particular site, high flows have passed over the top of bank, run down through an adjacent hayfield and rejoined the river at the next channel bend. The river is attempting to cut an oxbow which would create an island of about 30 acres of farmland.

Wilder Dam, a run-of-river hydropower project located about 38 miles downstream (river mile 217), creates a backwater effect at the Haverhill site under normal flow conditions. Water levels at the site, vary from 1 to 1.5 feet per day and range from elevation 386 to 383 feet NGVD over the weekly period due both to Wilder Dam operations and to the fluctuating flow releases from a series of upstream hydro-

power dams. During floodflows, the backwater effect of Wilder Dam diminishes and the hydraulic control becomes more nearly that of open channel characteristics. Water surface profiles for a range of flow conditions in the Wilder Dam pool are shown on Plate 3.

f. Environmental Considerations. The bank stabilization project is expected to result in a net improvement in wildlife habitat and water quality. The originally exposed riverbank was suited only as a seasonal nesting site for cliff swallows. The stabilized slope, revegetated above the protection structures with unmown grasses, legumes, vines and shrubs, provides beneficial habitat for a wider variety of wildlife. Bank stabilization and toe protection will also reduce siltation and localized turbidity in the Connecticut River.

6. Demonstration Site - Test Reach

a. Hydrologic Characteristics. The hydrologic characteristics are as previously stated. Hydrographs of average annual, annual peak and summertime flows for a 15-year period at the USGS gage at Wells River, Vermont are shown on Plate 4. A hydrograph of average daily discharges for the period of record is shown on Plate 5. Ice usually forms along the shore around mid-December (actual time of occurrence varies from year to year depending upon coldness of air temperature) and may or may not form an entire cover on the river. Photo 13 shows ice conditions at the project site in February 1981. The ice usually breaks up during the spring snowmelt runoff period in April.

b. Hydraulic Characteristics. As previously discussed in paragraph 5e, the demonstration site is situated in the upper reach of the Wilder Dam pool and is subject to regular daily and weekly water level and flow fluctuations due to both downstream and upstream hydropower operations (Plate 6). The stage-discharge rating curve is given on Plate 7. Field measurements made by the USGS near the toe of bank under normal flows ranging from 2,500 to 13,500 cfs indicate velocities less than 2 fps. Under floodflows in the order of 40,000 cfs local velocities have been calculated to be about 7 to 8 fps. Velocity distribution within the river channel cross section is unknown.

Wind generated waves are a relatively insignificant cause of

bank erosion at the demonstration site due to extremely limited fetch. Also, boat generated waves play a minor role compared to other causative hydraulic factors as there is no commercial navigation and only a small amount of recreational boating traffic.

c. Riverbank Description.

(1) Bank Materials. Materials composing the banks and valley floor of the Connecticut River are classified as silty, fine sands and fine sandy silt. Alluvial deposits in the vicinity of Haverhill, New Hampshire are comprised mainly of the Hadley soils. Analysis of test results from the borings indicate that bank materials at the project site are water-laid deposits composed mostly of silty, fine sand (SM) and fine, sandy silts (ML). Soil classification and test results are given in Plates 8 and 9.

(2) Description of Vegetation. Vegetation in the test reach consisted of a narrow band of agricultural, native and weed grasses and some shrubs. Most of the riverbank was severely eroded to an almost vertical earth face. Adjoining the riverbank is an agricultural field, seasonally plowed for row crops. No significant trees existed along the riverbank in the eroded area.

(3) Erosion Conditions. Fifty-four sites along the banks of the Connecticut River in the pool behind Wilder Dam (river miles 217 to 259) are eroding. Rates of erosion as high as 13 feet of bank depth in a year have been documented by the US Soil Conservation Service. The main types of erosion being observed are sloughing, undercutting, and mass wasting, in that order, and the principal cause is shear stress associated with high streamflow. Other causes such as pool fluctuation, seepage, boat waves and overbank drainage play lesser roles in the overall erosion process. At the Haverhill site, the shear force is magnified by its location at the outside of a sharp riverbend, a phenomenon that also affects about 33 percent of the other erosion sites in the Wilder pool. Erosion prior to construction of the bank protection is shown on Photos 1 and 2. No previous attempts at erosion control are known to exist in the demonstration site area.

III. DESIGN AND CONSTRUCTION

7. General. Five different and somewhat novel methods of streambank protection were used in the Haverhill demonstration project. These included four types of revetment - gabion mattress, sand-cement filled bags, used rubber tire mattress and baled hay mattress - and a reach with no toe protection and only vegetative upper bank protection. All four revetment panels included vegetative protection on the upper bank. The arrangement of the various test panels are shown on Plate 10.

8. Basis for Design.

a. Lower Bank Protection. The primary goal in selecting the types of protection to be utilized was to gain experience with new and innovative methods of streambank erosion control. Gabion mattresses and sand-cement filled riprap bags were selected because of their commercial availability and the New England Division's desire to gain field experience with them. Rubber tire matting and baled hay were selected on the basis that they are readily available materials and on the premise that they would require relatively simple construction techniques that local government agencies and private land owners could employ. The panel with no toe protection was selected to determine whether simply cutting the eroded bank back to a uniform slope would result in a stable section. In effect, the panel was to serve as a control against which to gage the necessity for and effectiveness of lower bank (toe) protection.

b. Upper Bank Protection. Upper bank protection was provided by a series of test areas of various mixes of grasses, legumes, vines and shrubs. In general, vegetation was used as an alternative to more expensive structural measures in the portion of the bank above the normal high water line, and also as a more natural appearing bank cover in the project's rural setting. Selection of plant species was based on knowledge of suitable native and adapted species types commercially available in the region. Additional technical assistance was provided by the Soil Conservation Service (SCS), US Department of Agriculture. Three methods of mulching were also compared: excelsior

fiber mats, hay with netting, and wood chips. Grasses were selected to provide a thick vegetative mat that protects the soil surface from the erosive effects of rainfall and high stage flows and to buffer the impact of floating debris, induce minor silt deposition and reinforce and stabilize the soil surface through extensive fine texture roots. Shrubs and vines, planted as container grown, healthy, young plants, provide less rapid soil protection but better wildlife cover and food potential.

Plates 10 and 11 list the plant species used and show their location in the project. In general, the experimental grass mixes were chosen on the basis of rapid establishment, suitability for growth with little or no maintenance, and adaptability to periodic inundation. Legumes were added to some mixes for supplemental nitrogen. Shrub species were chosen for wildlife value and adaptability to periodic inundation. Vines were chosen for their hardiness, woody growth and ability to provide rapid low cover.

9. Construction Details. The total project length is 2600 feet. Five techniques of bank protection, described below and shown on Plates 10 and 12 were installed in approximately 500-foot reaches along the bank.

a. Reach 1. Gabions, 3 feet wide by 12 feet in length were wire laced together and anchored to the slope in order to form a twelve-inch thick protective rock mattress. The mattress was constructed from an underwater dumped gravel toe on a 1.75H to 1V graded slope to a point 3 feet vertically above the normal water line. Filter fabric (Typar Spunbonded Polypropylene, Style 3421) was used under the gabions along this reach. The bank above was dressed to its natural slope (1.5H to 1V) and seeded. See Photo 3.

b. Reach 2. A matting of interlocked used rubber tires was placed on the underwater slope from the toe up to a point 3 feet vertically above the normal water line. The tires were placed on a 1.25H to 1V slope and anchored to the bank. Anchoring was accomplished using a steel reinforcing rod driven into the bank at the center of the tire, which was then filled with concrete. This was done only at specific tires spaced at 12-foot intervals along the top and bottom of the

matting. Tires in the first half of the reach (2A) were filled with crushed stone while tires in the remainder of the reach (2B) were filled with random fill material. The bank above was dressed to its natural slope (1.25H to 1V) and vegetated. See Photos 4 and 5.

c. Reach 3. Sand and cement filled reinforced paper riprap bags (Hudson's Reinforced Paper Riprap Bags) were placed against the bank on a 2H to 1V slope from the underwater toe up to a point 3 feet above the normal waterline. Filter fabric was placed under the sand and cement riprap bags along this reach. The upper bank was formed to a 2H to 1V slope and vegetated. See Photo 6.

d. Reach 4. The bank was reformed to a 2H to 1V slope and overlaid with baled hay which is contained by an anchored wire mesh. Filter fabric was used under the hay bales only in the first half of the reach (4A). The upper bank was formed to a 2H to 1V slope and vegetated. During construction a change was made substituting screw earth anchors for the specified smooth rod anchors in order to remedy the problem of flotation created by the buoyancy of the hay bales.

e. Reach 5. The final section remained in its original condition below the waterline. The upper bank was formed to a 2H to 1V slope and vegetated.

10. Costs. Total cost and unit cost on a linear foot basis for each of the test reaches in the Haverhill project are shown in the following table. These costs were developed based on contractor bid items and do not include 6 and 7 percent markups for Corps of Engineers engineering and design, and supervision and administration, respectively.

Test Reach	Total Cost (dollars)	Cost Per Linear Foot (dollars)
1	47,500	95
2A	13,600	54
2B	13,800	55
3	50,000	100
4A	25,000	100
4B	24,100	96
5	22,200	44

Costs are for the time of project construction which was completed in September 1979. The cost of gabion protective dividers has not been included in the cost of each reach. Total contract cost for the entire project including all facets was about $250,000.

IV. PERFORMANCE OF PROTECTION

11. Monitoring Program. The elements of the monitoring program are summarized on Plate 13. The site was monitored for one year beginning in November 1979, just after completion of construction. Monitoring was discontinued after one year due to inadequate funding. Baseline topographic surveys were taken the year before and just prior to construction. Additional cross-section surveys of Reach 5 (which failed) and of some minor washouts caused by overland runoff were taken in the fall of 1980 (Plate 14 and Photo 12). Settlement monuments were also checked at this time to determine if any change had occurred since post construction readings were taken; no significant change was noted. No significant floods have occurred during the monitoring period; the Connecticut River has not reached bankfull conditions.

Velocity measurements were taken by the US Geological Survey during three visits to the site. These readings, taken during low, moderate, and higher flows experienced since construction, are shown on Plate 15. Visual inspections of the project were conducted by NED in the fall of 1979 and in spring and fall 1980. Pertinent findings were noted in periodic inspection reports which were submitted to the Waterways Experiment Station. Ground level photographs were taken during these inspections (Photos 7 through 12) to document changed conditions. Additionally, CRREL inspected the site in February 1980 to observe and report on winter ice conditions (Photo 13). Material tests of bank soils (Plates 8 and 9) were accomplished prior to construction and materials used in construction were tested on an as-needed basis.

12. Evaluation of Protection Performance.

a. Lower Bank Protection. The maximum discharge experienced to date by the completed project occurred on 11 April 1980 and was estimated to be about 19,400 cfs (which is substantially less than the mean annual flood (33,100 cfs). The lower bank protection at the site was overtopped by about 3 feet during this flow. Some minor scouring was noted in the zone of overtopping on the vegetated upper bank, especially where shrubs with wood chips had been placed. Reach 5, which has no toe protection, suffered considerable erosion (Photo 12). Some of the earth filling in the rubber tires in Reach 2B has washed away with minor settlement being noted. Additionally, some rusting was noted on metal tire strapping below the water surface. Limited bulging and voids were observed in the baled hay protection. Sand-cement filled bags are deteriorating as intended, however, the sand-cement itself is starting to erode. Gabion protection and stone filled auto tires were found to be in good condition. Overall, the protective systems are performing satisfactorily with the exception of nontoe-protected panel (Reach 5) and the earth filled used tire matting (Reach 2B).

b. Upper Bank Protection. Some limited gully erosion on the upper bank, due to concentrated surface runoff from the adjacent farm field, has occurred but it poses no serious threat to the overall project. Since no significant river flows have occurred following completion of project construction and planting of upper bank protection, complete evaluation of the effectiveness of various plants in controlling erosion is not possible. The vegetation has become well established and has effectively acted to reinforce the soil surface and resist rainfall. However, the long term effectiveness of the protective works will not be known until significantly large river flows occur.

13. Rehabilitation. No rehabilitation has taken place at the project. However, plans and specifications are being developed which include the use of 12-inch thick gabion mattresses to provide lower bank protection in Reach 5 (no toe protection), which has failed. Repair

work consisting of grouted rock gutter drains is planned to correct minor gulley erosion caused by concentrated surface runoff at two locations. If the loss of earth fill in the used auto tires continues, some stabilizing action in that reach may be necessary in the future.

14. Conclusions. Preliminary evaluations in the absence of experienced high flow conditions indicate several general conclusions.

a. Lower Bank Protection. Toe protection is an essential ingredient of any successful streambank protection. This is evidenced by the failure of Reach 5, which was the only one constructed without toe protection. Stone fill should be used in rubber tire mattress instead of random earth filling. This conclusion is based on the superior performance of Reach 2A. No performance difference was observed between reaches with and without filter cloth. Noncorrodable banding should be utilized in rubber tire mattresses. Additionally, a method for controlled conveyance of overland surface runoff to the river should be provided.

b. Upper Bank Protection. Seed cover on upperbank is best established in the spring, so that a full growing season is available for root development and top growth. Fall planting requires a larger percentage of winter rye or other nurse grass to control erosion until the perennial grasses become well established. Mulching grass seeded areas with hay and plastic netting works as well if not better than the commercial excelsior matting. The added per square foot cost of nursery grown shrubs and vines may not justify their use for erosion control, as they take too long to establish adequate cover, and the root systems do not develop the fine network of roots adequate to control surface erosion. Wood chips or bark mulch, commonly used to control surface erosion and retain soil moisture around plants, are not reliable on a 2:1 riverbank slope; high water rapidly washes mulch away, leaving the soil surface exposed to riverflow or overbank erosion. Shrub planting areas at the top of bank have not shown any soil or mulch erosion.

Short term performance evaluation indicates that Siberian Dogwood, Cornus alba siberica, Redosier Dogwood, Cornus stolonifera and American

Bittersweet, Celastrus americana provided the best growth and cover of the shrubs and vines used. Of the seed mixes, the best cover has been provided by mixtures #1 and #3, both dominated by Reed Canary grass, Phalaris arundinacea and Kentucky 31 Fescue, Festuca elatior arundinacea (Plate 11).

PHOTO 1
AERIAL VIEW FROM WEST
APRIL 1978

PHOTO 2
BOAT VIEW FROM WEST
MAY 1977

HAVERHILL PROJECT
PRE-CONSTRUCTION

PHOTOS 1 AND 2

PHOTO 3
REACH 1
JULY 1979

PHOTO 4
REACH 2A
JULY 1979

HAVERHILL PROJECT
DURING CONSTRUCTION

PHOTOS 3 AND 4

PHOTO 5
REACH 2B
JULY 1979

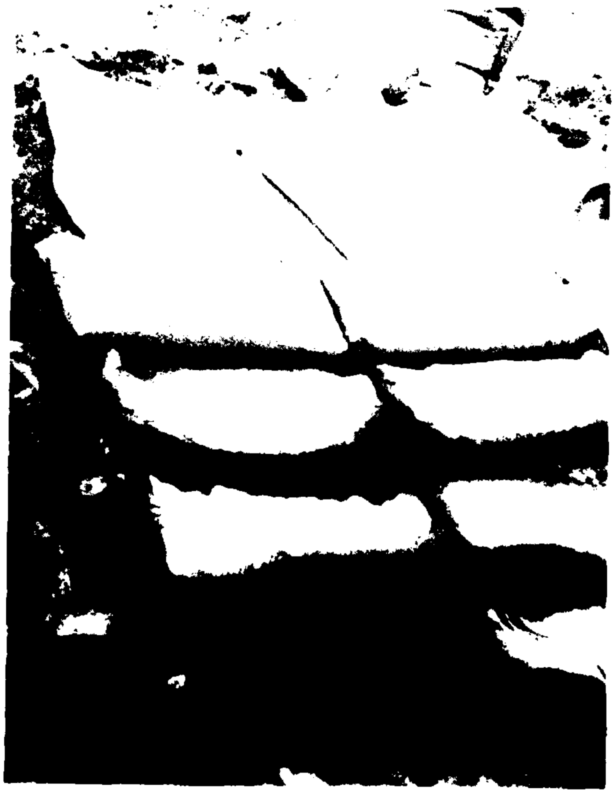


PHOTO 6
REACH 3
JULY 1979

HAVERHILL PROJECT
DURING CONSTRUCTION

PHOTOS 5 AND 6

PHOTO 7
REACH 1
MAY 1980

PHOTO 8
REACH 2B
MAY 1980

HAVERHILL PROJECT
POST CONSTRUCTION

PHOTOS 7 AND 8

PHOTO 9
REACH 2B
MAY 1980

PHOTO 10
REACH 3
MAY 1980

HAVERHILL PROJECT
POST CONSTRUCTION

PHOTOS 9 AND 10

PHOTO 11
REACH 4
MAY 1980

PHOTO 12
REACH 5
MAY 1980

HAVERHILL PROJECT
POST CONSTRUCTION

PHOTOS 11 AND 12

ICE CONDITIONS
20 FEBRUARY 1981
VIEW LOOKING UPSTREAM

PHOTO 13

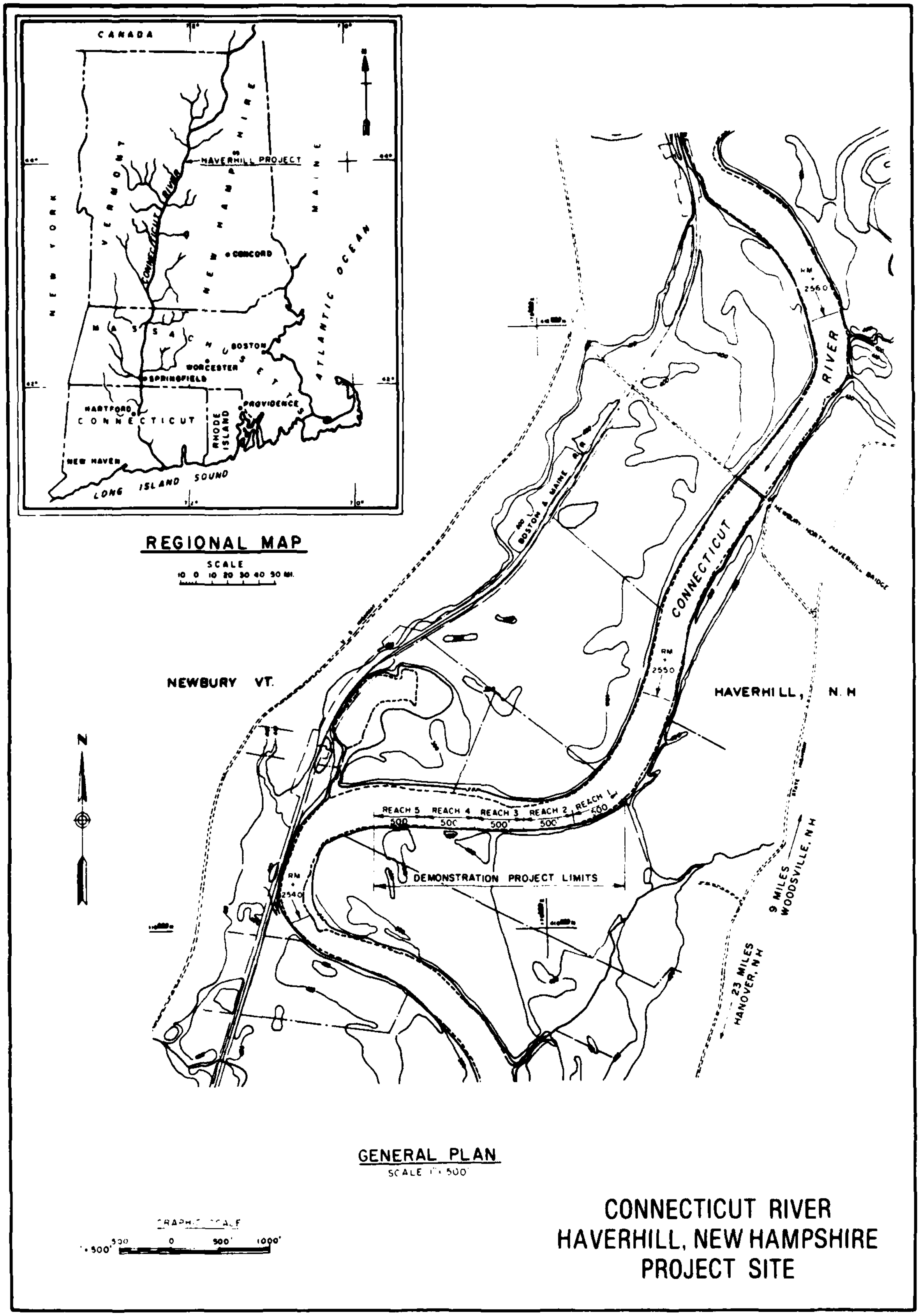

CANADA
HAVERHILL PROJECT
NEW YORK
VERMONT
NEW HAMPSHIRE
MAINE
CONNECTICUT RIVER
CONCORD
ATLANTIC OCEAN
MASSACHUSETTS
BOSTON
WORCESTER
SPRINGFIELD
HARTFORD
CONNECTICUT
RHODE ISLAND
PROVIDENCE
NEW HAVEN
LONG ISLAND SOUND
REGIONAL MAP
SCALE
10 0 10 20 30 40 50 MI.
CONNECTICUT RIVER
BOSTON & MAINE R.R.
NEWBURY NORTH HAVERHILL BRIDGE
RM 2560
RM 2550
RM 2540
NEWBURY VT.
HAVERHILL, N.H
N
REACH 5
REACH 4
REACH 3
REACH 2
REACH 1
500'
400'
DEMONSTRATION PROJECT LIMITS
9 MILES WOODSVILLE, N.H
23 MILES HANOVER, N.H
GENERAL PLAN
0 500' 1000'
CONNECTICUT RIVER
HAVERHILL, NEW HAMPSHIRE
PROJECT SITE

PLATE 1

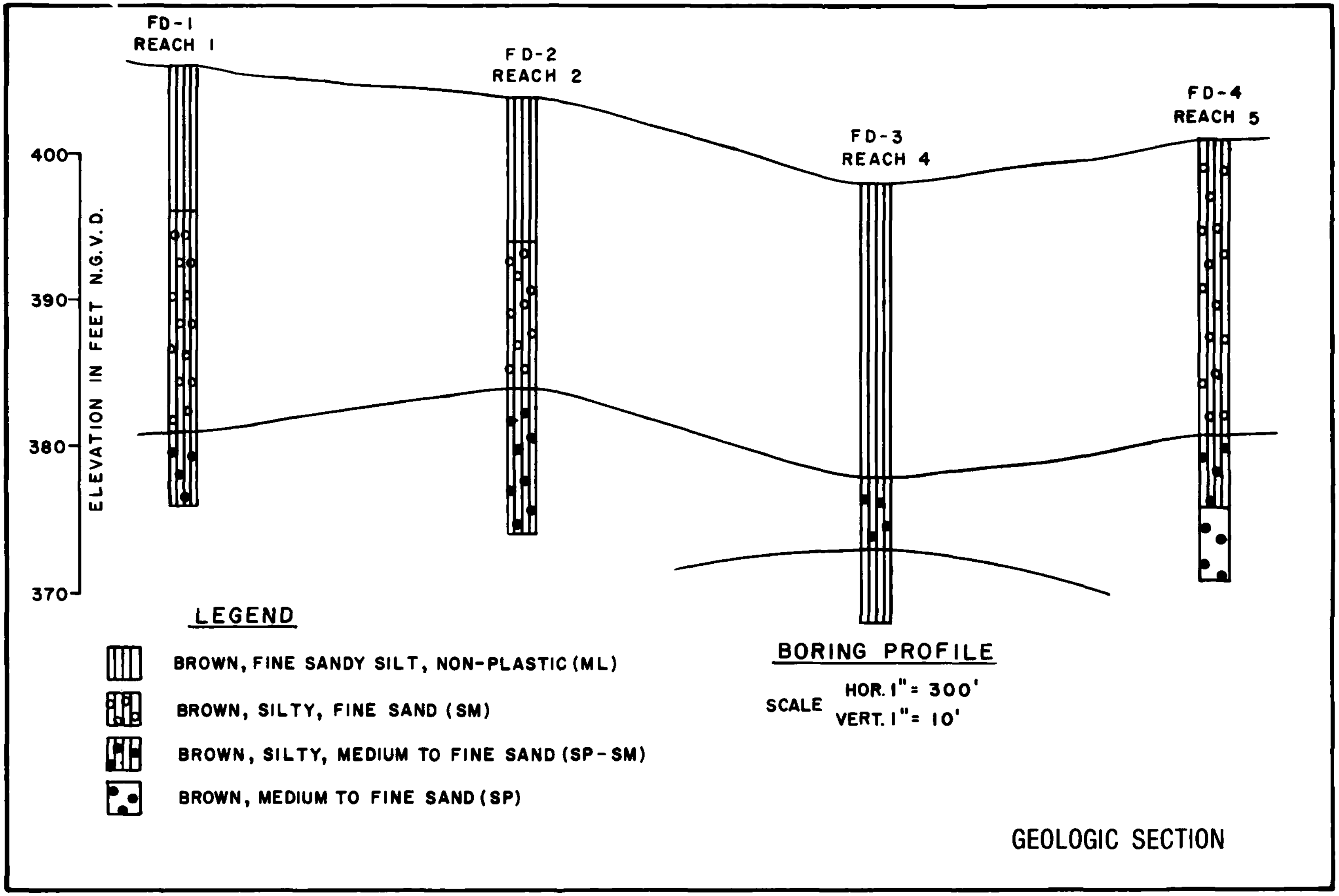

FD-1
REACH 1
FD-2
REACH 2
FD-3
REACH 4
FD-4
REACH 5
400
390
380
370
ELEVATION IN FEET N.G.V.D.
LEGEND
BROWN, FINE SANDY SILT, NON-PLASTIC (ML)
BROWN, SILTY, FINE SAND (SM)
BROWN, SILTY, MEDIUM TO FINE SAND (SP-SM)
BROWN, MEDIUM TO FINE SAND (SP)
BORING PROFILE
SCALE HOR. 1" = 300'
VERT. 1" = 10'
GEOLOGIC SECTION

PLATE 2

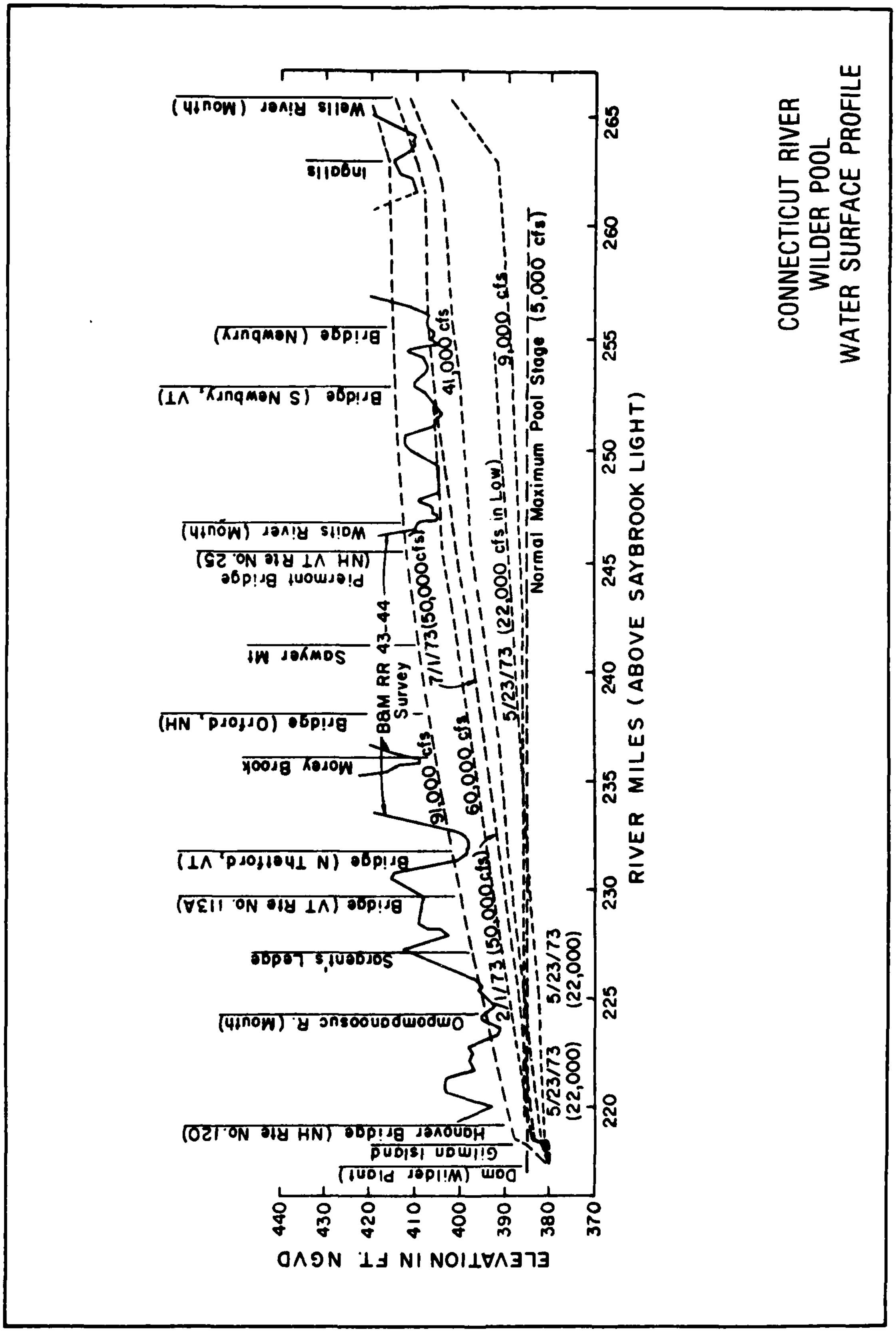

CONNECTICUT RIVER
WILDER POOL
WATER SURFACE PROFILE
ELEVATION IN FT. NGVD
RIVER MILES (ABOVE SAYBROOK LIGHT)
440
430
420
410
400
390
380
370
220
225
230
235
240
245
250
255
260
265
Dam (Wilder Plant)
Gilman Island
Hanover Bridge (NH Rte No. 120)
Ompompanoosuc R. (Mouth)
Sargent's Ledge
Bridge (VT Rte No. 113A)
Bridge (N Thetford, VT)
Morey Brook
Bridge (Orford, NH)
Sawyer Mt
Piermont Bridge
(NH VT Rte No. 25)
Waits River (Mouth)
Bridge (S Newbury, VT)
Bridge (Newbury)
Ingalls
Wells River (Mouth)
B&M RR 43-44
Survey
91,000 cfs
60,000 cfs
41,000 cfs
9,000 cfs
7/1/73 (50,000 cfs)
2/1/73 (50,000 cfs)
5/23/73 (22,000 cfs in Low)
Normal Maximum Pool Stage (5,000 cfs)
5/23/73
(22,000)
5/23/73
(22,000)

PLATE 3

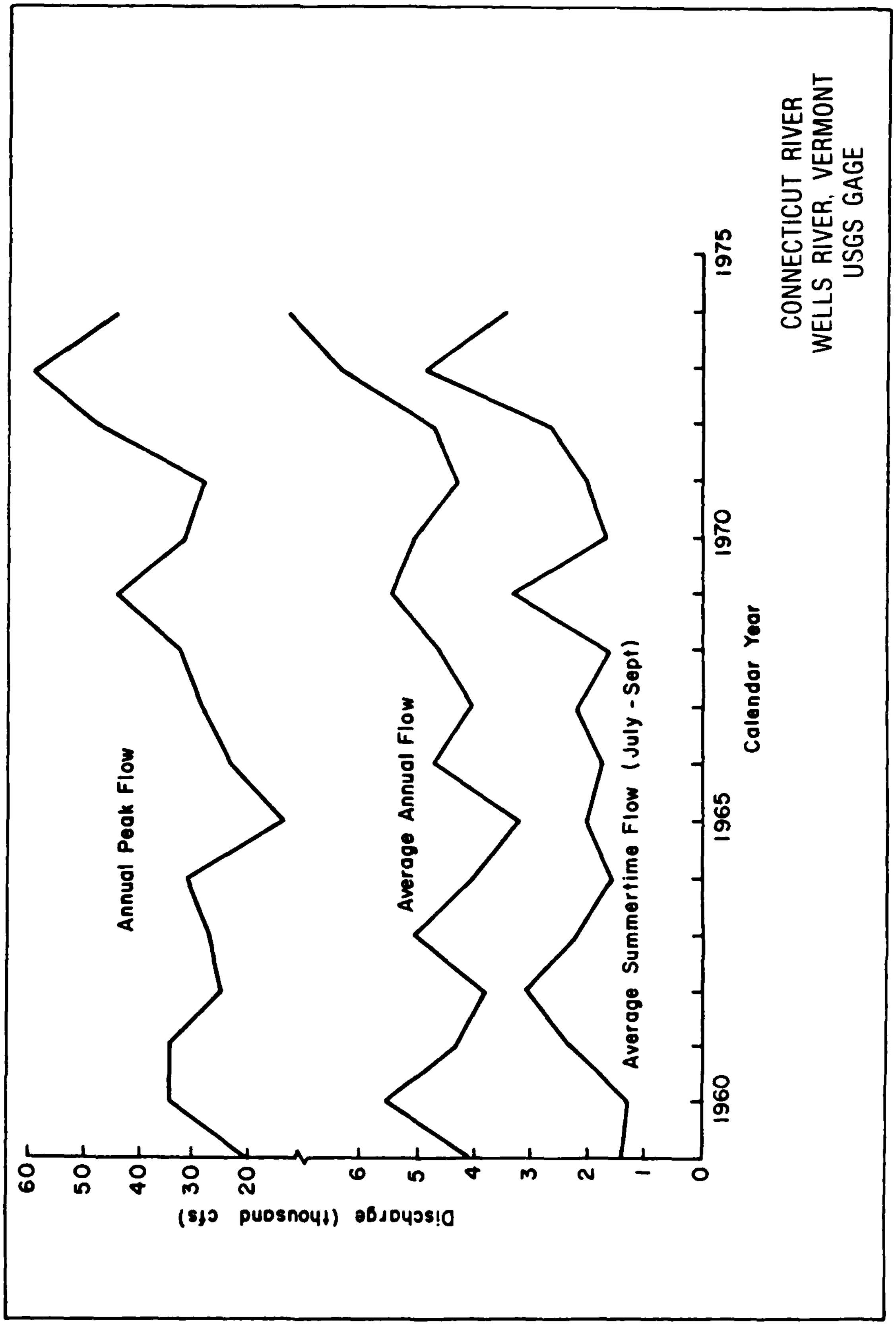
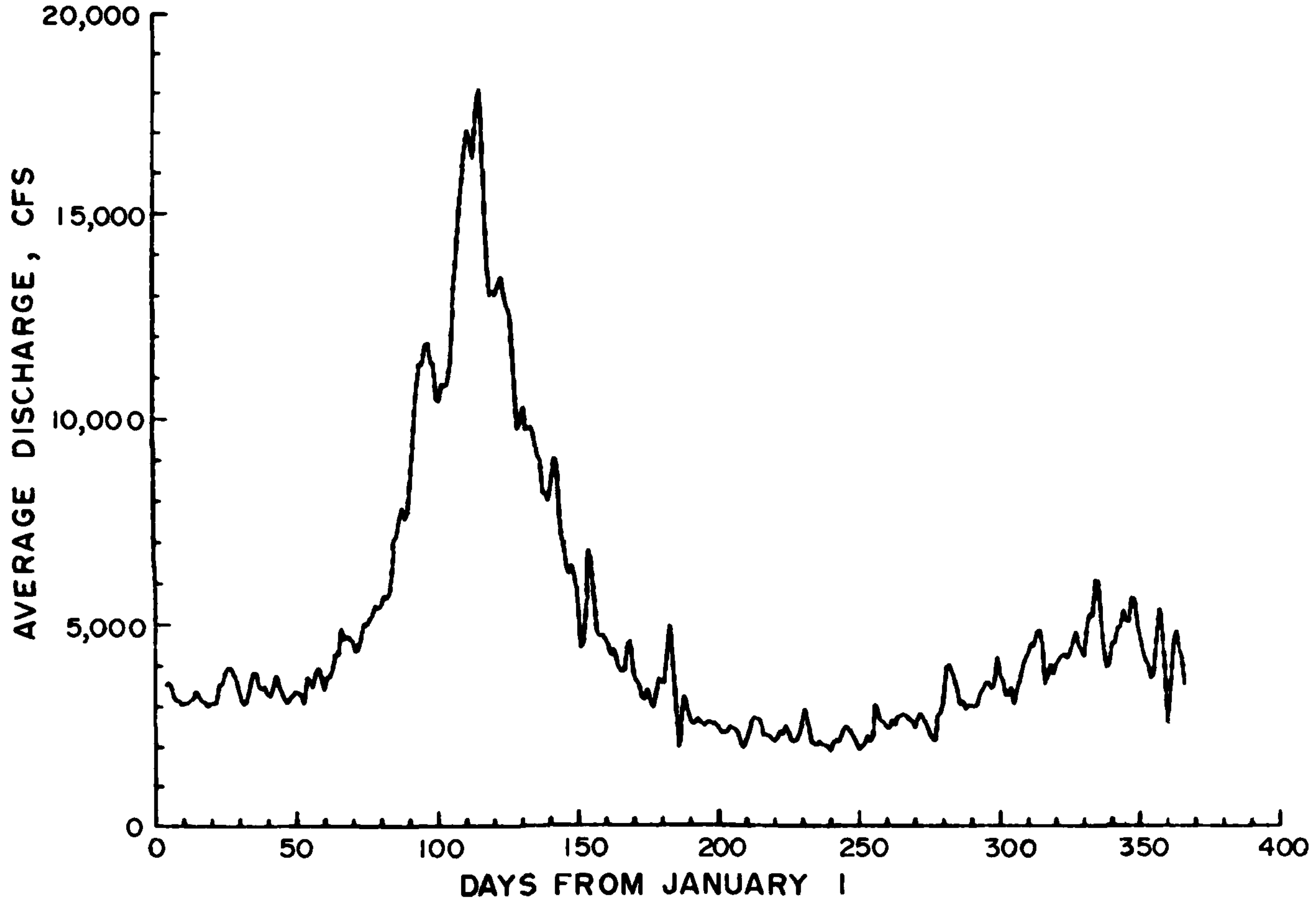
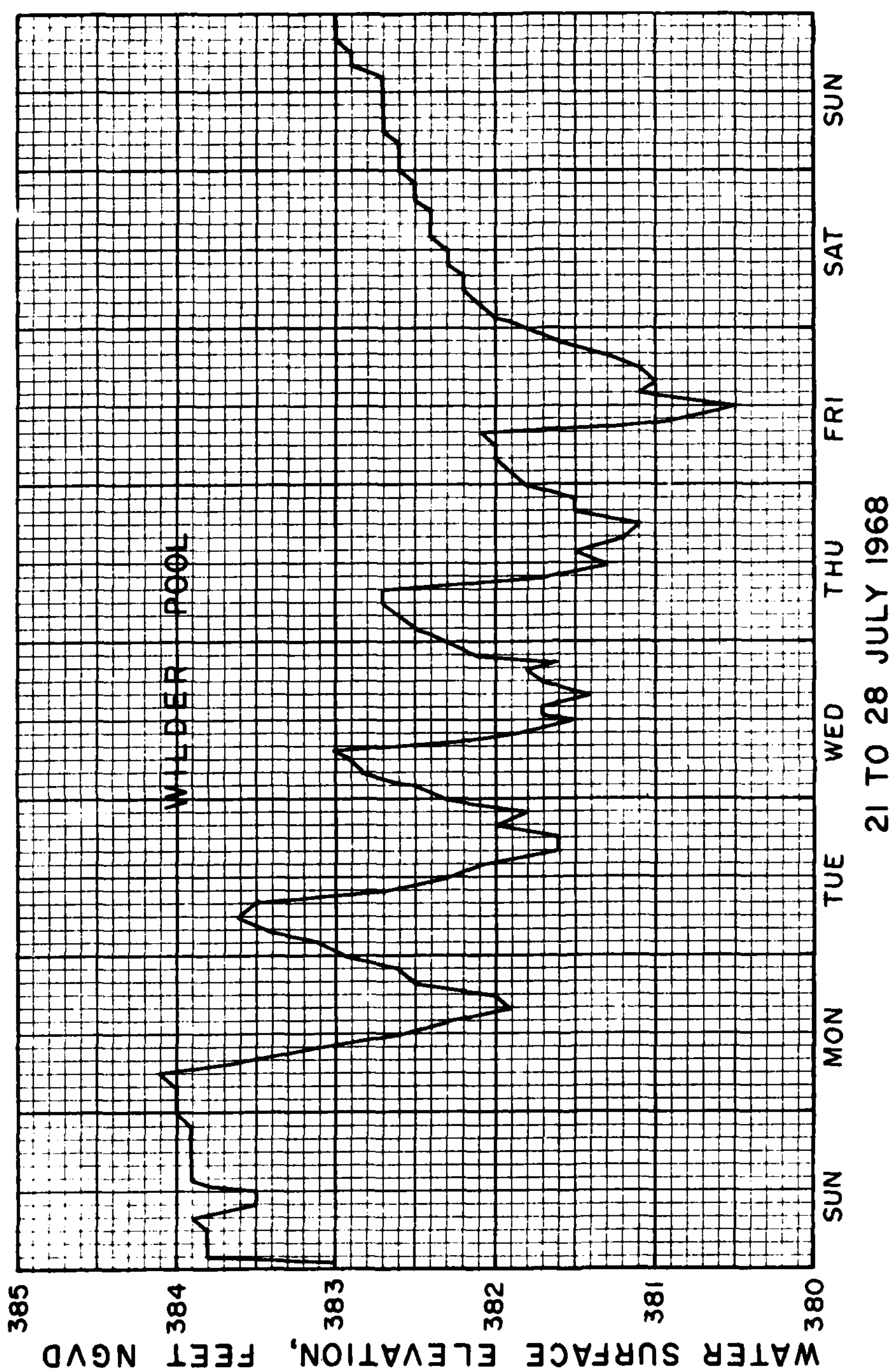

CONNECTICUT RIVER
WELLS RIVER, VERMONT
USGS GAGE
Annual Peak Flow
Average Annual Flow
Average Summertime Flow (July - Sept)
Discharge (thousand cfs)
Calendar Year
60
50
40
30
20
6
5
4
3
2
1
0
1960
1965
1970
1975

PLATE 4



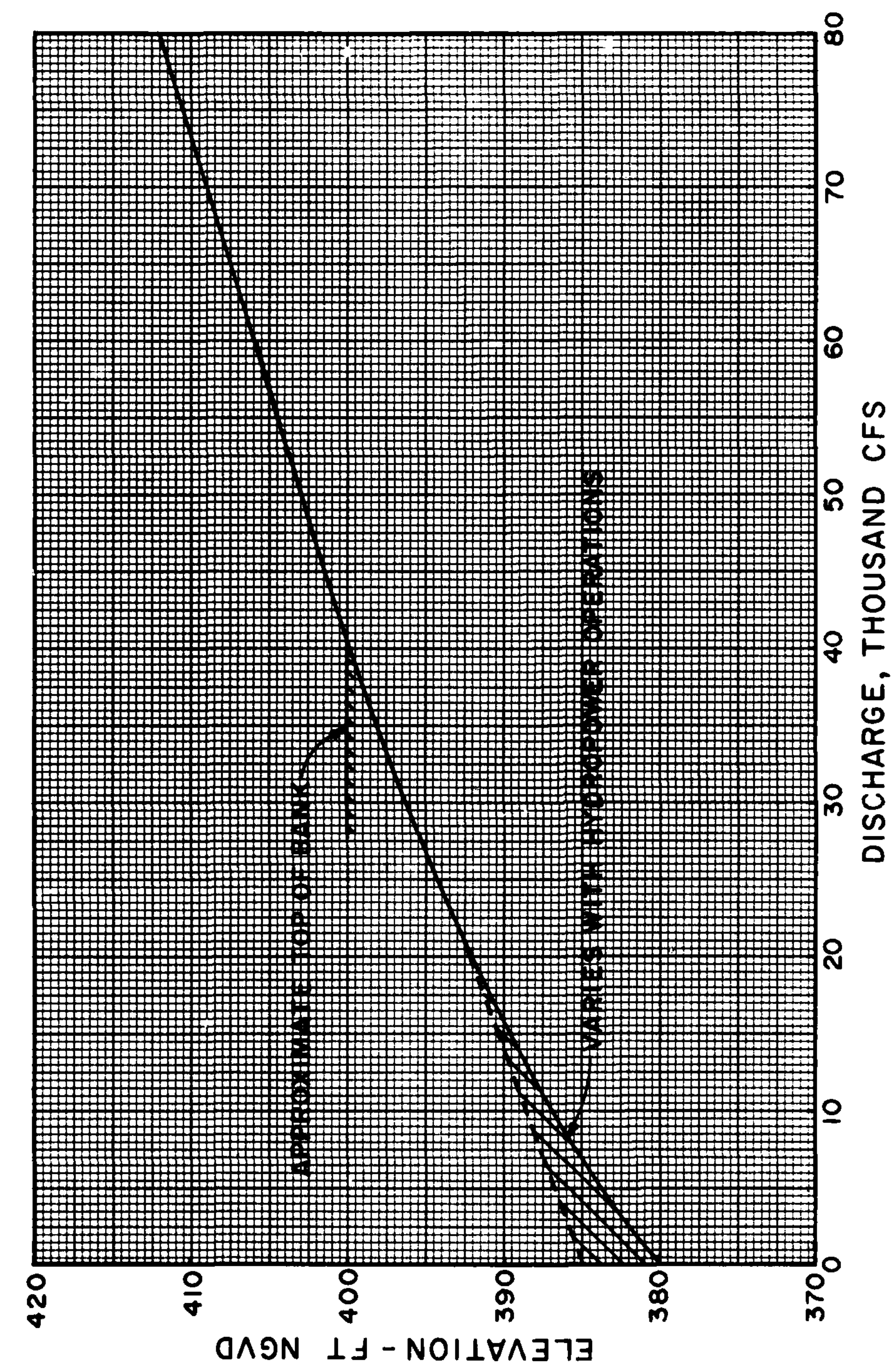
CONNECTICUT RIVER
WELLS RIVER, VERMONT
AVERAGE ANNUAL HYDROGRAPH

PLATE 5



WILDER POOL FLUCTUATIONS

PLATE 6



APPROXIMATE STAGE-DISCHARGE RELATIONSHIP
HAVERHILL DEMONSTRATION PROJECT

PLATE 7

REACH NO.	EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS: GRAVEL %	MECHANICAL ANALYSIS: SAND %	MECHANICAL ANALYSIS: FINES %	MECHANICAL ANALYSIS: D_{10} mm.	ATT. LIMITS: LL	ATT. LIMITS: PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT: TOTAL	NAT. WATER CONTENT % DRY WT: -NO4
1	FD-1	406±	J-1	0.0- 5.0	ML	0	32	68	0.015	NP	NP	2.73	16.9	16.9
			J-3	10.0-15.0	SM	0	67	33	0.035	NP	NP	2.68	11.4	11.4
			J-6	25.0-30.0	SP-SM	3	89	8	0.10	NP	NP	2.71	16.3	-
2	FD-2	404±	J-1	0.0- 1.5	ML	0	20	80	0.0066	34	29	2.68	27.0	27.0
			J-3	11.5-15.0	SM	0	58	42	0.026	NP	NP	2.65	10.3	10.3
			J-6	26.5-30.0	SP-SM	0	94	6	0.10	NP	NP	2.70	21.9	21.9
4	FD-3	398±	J-1	0.0- 5.0	ML	0	38	62	0.018	NP	NP	2.68	16.6	16.6
			J-3	10.0-15.0	ML	0	25	75	0.012	NP	NP	2.72	34.0	34.0
			J-5	20.0-25.0	SP-SM	3	89	8	0.080	NP	NP	2.68	21.6	-
			J-6	25.0-30.0	ML	0	5	95	0.014	NP	NP	2.71	27.5	27.5
5	FD-4	401±	J-1	0.0- 1.5	SM	0	76	24	0.040	NP	NP	2.68	13.5	13.5
			J-4	15.0-16.5	SM	0	78	22	0.040	NP	NP	2.69	29.6	29.6
			J-6	25.0-26.5	SP	0	96	4	0.15	NP	NP	2.69	25.7	25.7

SOIL CLASSIFICATION DATA

PLATE 8

GRADATION CURVES
EXISTING BANKS MATERIALS

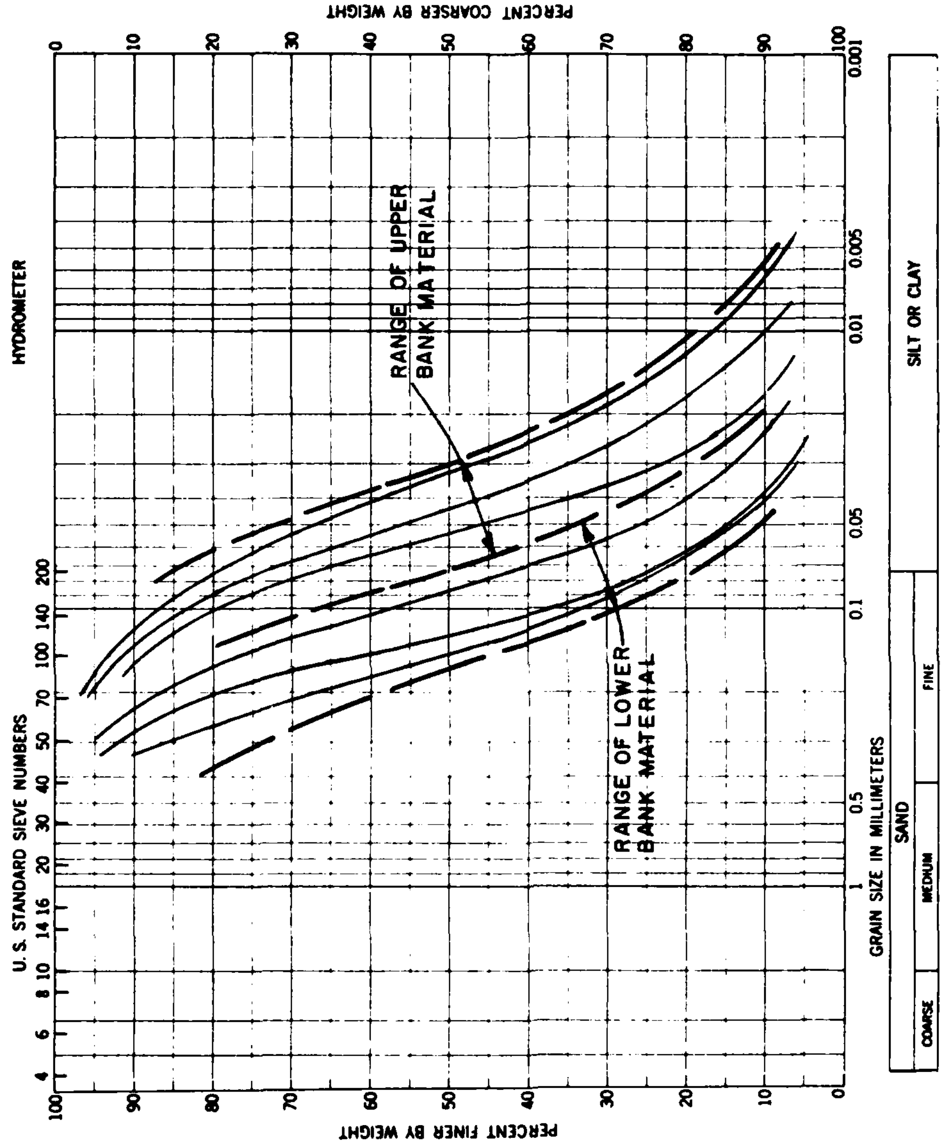
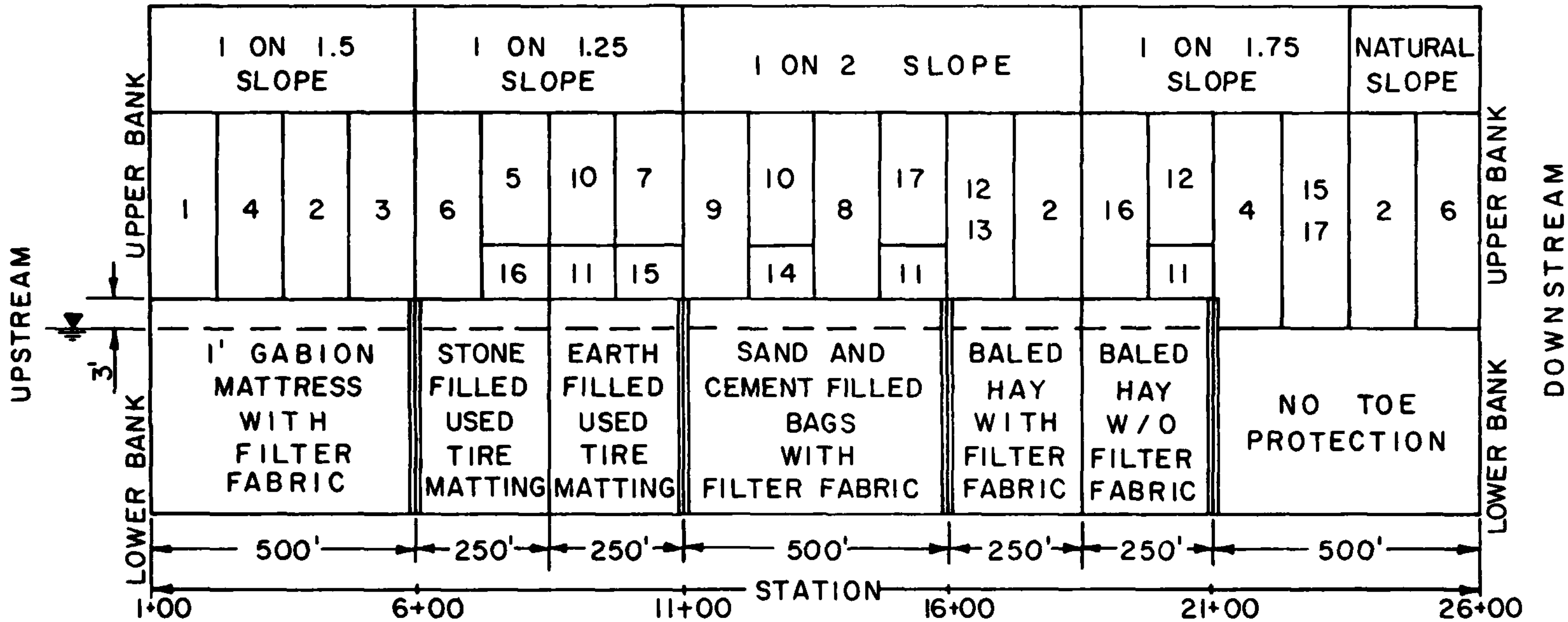


PLATE 9

NOTE:
GABION PROTECTIVE DIVIDERS
LOCATED AT STA'S. 6+00, 11+00,
16+00 AND 21+00.



SEE PLATE 11 FOR LIST OF UPPER BANK VEGETATION TYPES

CONNECTICUT RIVER
HAVERHILL, NEW HAMPSHIRE
PROTECTIVE MEASURES

PLATE 10

SEED MIXTURES

1 REED CANARYGRASS
KENTUCKY 31 FESCUE
REDTOP

2 REED CANARYGRASS
CREEPING RED FESCUE
REDTOP

3 REED CANARYGRASS
KENTUCKY 31 FESCUE
BIRDSFOOT TREFOIL

4 CROWNVETCH
KENTUCKY 31 FESCUE
CREEPING RED FESCUE

5 FLAT PEA
KENTUCKY 31 FESCUE

6 CROWNVETCH
FLAT PEA
KENTUCKY 31 FESCUE

7 BIRDSFOOT TREFOIL
CREEPING RED FESCUE

8 CREEPING RED FESCUE
BIRDSFOOT TREFOIL
REDTOP

9 KENTUCKY 31 FESCUE
CREEPING FESCUE
REED CANARYGRASS
REDTOP

10 KENTUCKY 31 FESCUE
CREEPING RED FESCUE
REDTOP

SHRUBS AND VINES

11 PURPLEOSIER WILLOW

12 SIBERIAN DOGWOOD

13 REDOSIER DOGWOOD

14 SUMMERSWEET

15 AMERICAN BITTERSWEET

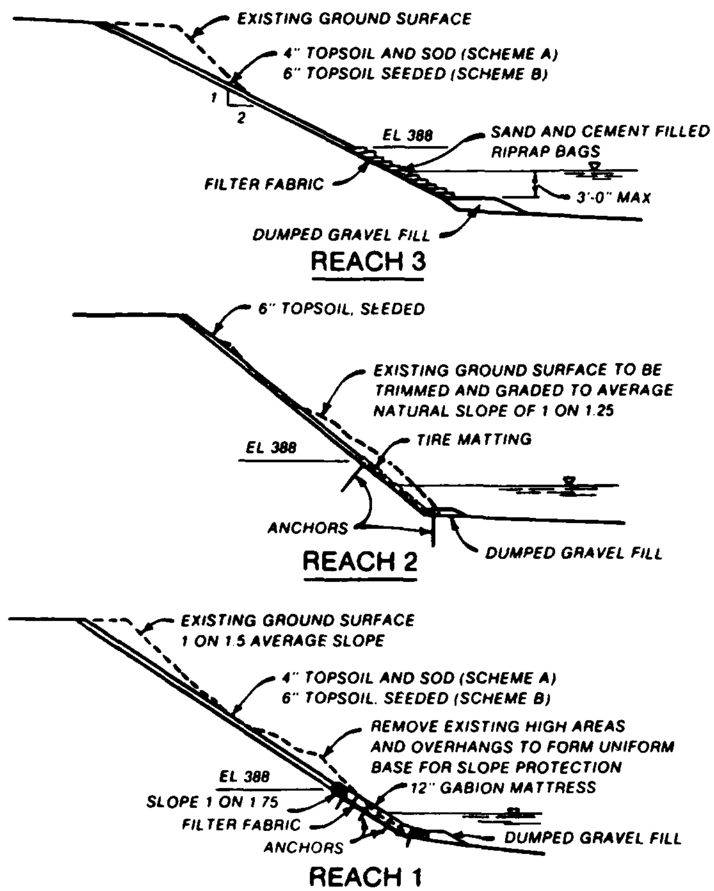
16 VIRGINIA CREEPER

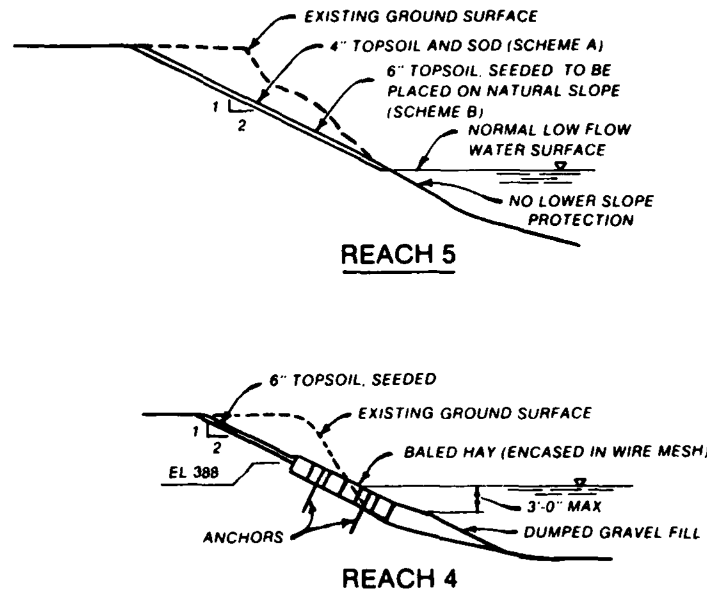
17 HALL'S HONEYSUCKLE

NOTE: NUMBERS ARE KEYED TO PANELS ON PLATE 10

UPPER BANK VEGETATION
HAVERHILL DEMONSTRATION PROJECT

PLATE 11





NOTE WHERE SCHEMES A AND B ARE INDICATED ON SECTIONS SCHEME A INDICATES THE FIRST 250' OF A PARTICULAR REACH AND SCHEME B INDICATES THE OTHER 250'

SCALE
5 0 5 10 FT

TYPICAL SCHEMES

CONNECTICUT RIVER
HAVERHILL, NEW HAMPSHIRE
PROTECTION SCHEMES

PLATE 12

FIELD DATA OF PHYSICAL FEATURES	FREQUENCY
1. Detailed topographic survey of bankline from thalwag to 15 feet beyond top of bank.	Taken the year before and just prior to construction.
2. Cross sections of bankline extending from thalwag to 15 feet beyond top of bank.	Developed for project design and taken as needed for repair and reconstruction work.
3. Settlement monuments checked for vertical and horizontal movements.	Checked just after construction and then on as needed basis.
4. Velocity measurements at toe of slope and 40 feet riverward and recording of discharge and water surface elevation.	Taken during low, medium, and higher flow periods.
VISUAL OBSERVATIONS	
1. Aggradation-degradation processes.	Semi-annually for all visual observations.
2. Erosion and river conditions.	
3. Changes in aquatic and terrestrial habitat.	
4. Changes in upper slope vegetation.	
5. Changes in structure integrity and material durability.	
6. Surface current flow patterns.	
MATERIAL TESTS	
1. Borings of bank material.	Once during preconstruction period.
2. Mechanical analysis of riverbank material.	
3. Classification of bank material.	
4. Specific gravity, Atterberg limits, hydrometers and water content of bank material.	
5. Analysis of construction materials when appropriate.	As needed during construction.
PHOTOGRAPHY	
1. Aerial photography	Preconstruction photos taken at site.
2. Ground level periodic photographs taken at predetermined locations.	Taken semi-annually beginning at the completion of construction.

CONNECTICUT RIVER
HAVERHILL DEMONSTRATION SITE
MONITORING PROGRAM

PLATE 13

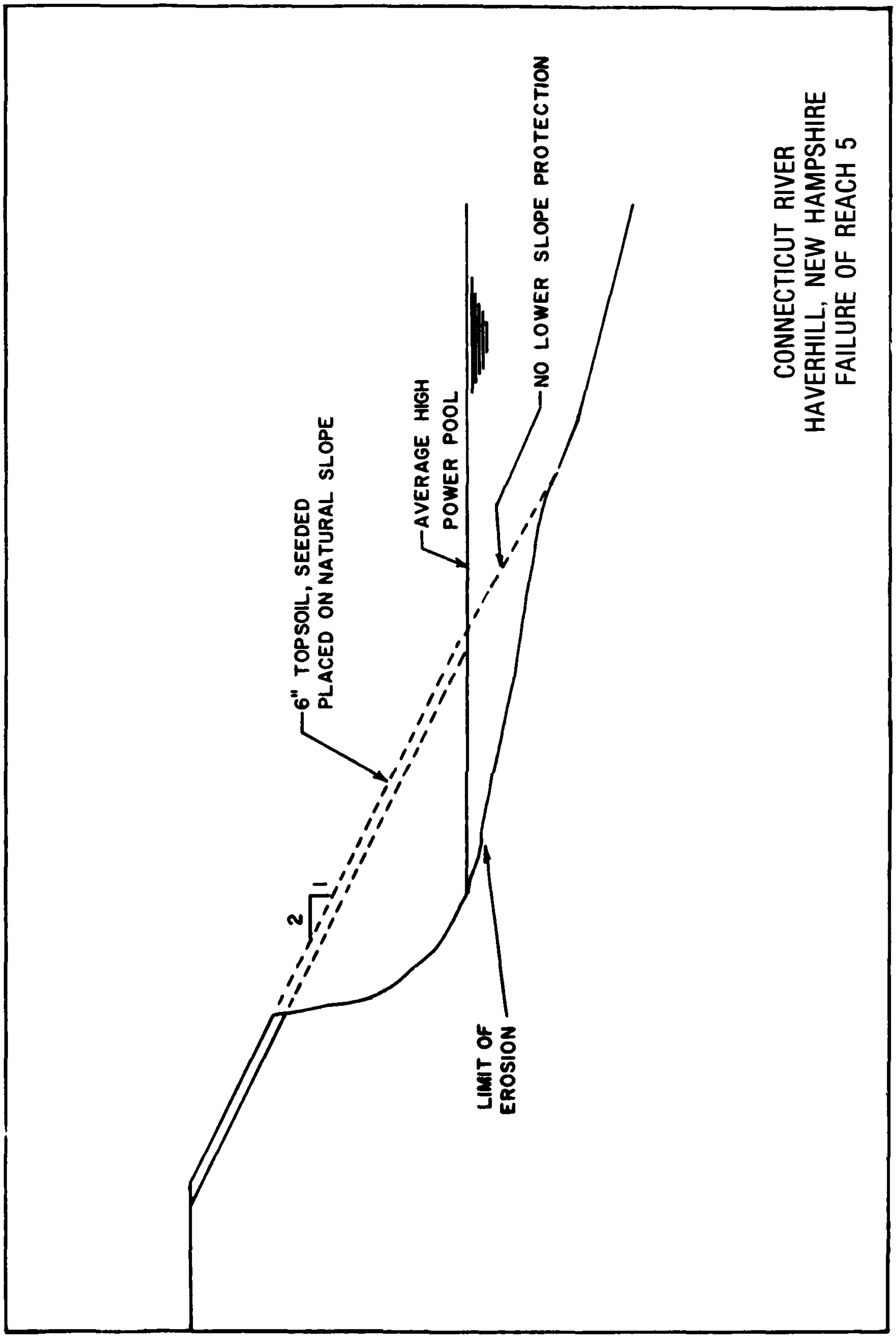

6" TOPSOIL, SEEDED
PLACED ON NATURAL SLOPE
AVERAGE HIGH
POWER POOL
NO LOWER SLOPE PROTECTION
2
1
LIMIT OF
EROSION
CONNECTICUT RIVER
HAVERHILL, NEW HAMPSHIRE
FAILURE OF REACH 5

PLATE 14

	Visit #1				Visit #2				Visit #3			
Date:	4 Dec 79				4 Apr 80				15 Apr 80			
Water Surface Elevation (Ft. NGVD):	383.5				385.0				390.1			
Estimated Discharge (cfs):	2,400				2,700				13,400			
	At Toe		40 Ft. From Toe		At Toe		40 Ft. From Toe		At Toe		40 Ft. From Toe	
Reach	Depth (ft)	Mean Velocity (fps)	Depth (ft)	Mean Velocity (fps)	Depth (ft)	Mean Velocity (fps)	Depth (ft)	Mean Velocity (fps)	Depth (ft)	Mean Velocity (fps)	Depth (ft)	Mean Velocity (fps)
1	1.5	0.39	12.8	0.76	3.8	0.53	12.6	0.90	9.5	1.95	19.5	2.90
2	1.2	0.25	11.0	1.04	2.5	0.55	13.2	1.24	6.3	1.99	15.1	3.32
3	1.4	0.39	6.5	1.12	2.5	0.46	7.6	1.14	6.0	1.50	13.9	2.64
4	1.8	0.34	10.4	1.36	1.6	0.38	11.0	1.60	6.3	1.30	16.1	2.68
5	1.2	0.18	11.5	1.40	2.5	1.35	12.5	1.38	11.5	2.32	17.2	2.92

CONNECTICUT RIVER
HAVERHILL DEMONSTRATION SITE
VELOCITY MEASUREMENTS

PLATE 15

CONNECTICUT RIVER AT
NORTHFIELD, MASSACHUSETTS

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

CONNECTICUT RIVER AT NORTHFIELD, MASSACHUSETTS
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Northfield Demonstration Site, Connecticut River, Northfield, Massachusetts. Location map shown on Plate 1.
2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.
3. Purpose and Scope. This report describes a demonstration project constructed by the New England Division to experiment with innovative techniques of streambank erosion control. It presents a description of the bank erosion problem and types of protection used. At the time of this writing construction of the project was just completed, and there has been no experience on which to base any performance evaluation.
4. Problem Resume. A 2000-foot reach of the left (east) bank of the Connecticut River just downstream from the Route 10 highway bridge is actively eroding. The bank varies from 20 to 25 feet in height above its toe and is eroding at the rate of between 1 and 3 feet per year. The erosion site is located within the Turners Falls Dam hydropower pool and is subject to widely fluctuating water surface levels due to both upstream and downstream hydropower operations on the river. Annual spring high water inundates the lower bank and the whole bank is inundated by unusually high spring runoff or extreme flood flows. Erosion conditions are typical of those caused by streamflow along a steep bank comprised of alluvial soils. Pool fluctuations, seepage and boat waves play lesser roles in the overall erosion process at the site. The land being lost is fertile farmland of the mid-Connecticut River Basin floodplain.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The project site lies in the central portion of the Connecticut River Basin, the largest in New England. Draining in a southerly direction with its source in northern New Hampshire and mouth at Long Island Sound in Saybrook, Connecticut, the Connecticut River extends more than 400 miles. The valley of the Connecticut River in its northern reaches where it forms the border between northen Vermont and New Hampshire is bounded on the west by the Green Mountains and east by the White Mountains, with mountain peak elevations of over 4000 feet NGVD being quite common. In this reach the valley floor is relatively narrow and of gentle terrain. Proceeding downstream toward the project site (approx. river mile 133) the valley floor widens with wide and extensive flood plains located in various reaches along the main stem. During major floods these areas act as large detention reservoirs which significantly reduce peak discharges at the project site. The lower mountain peaks (generally not over 2000 feet NGVD) of the southern Green Mountains of Vermont and Monadnock range of New Hampshire form the valley wall in the river reach just upstream from the project site. The normal stream gradient at the Northfield site is about 0.1 foot per mile.

b. Geology. The geology of the Connecticut River Basin can be subdivided into two distinct periods: prior to and following continental glaciation. Preglacial history of the Connecticut River is quite diverse. Bedrock of the area consists of heavily folded and faulted metamorphic and igneous rocks. The metamorphic rocks include phyllites, schists, and gneisses. The igneous bodies are granite, granodiorite, and quartz monzanite with occasional intrusions of volcanic materials. The trends of major structural features in Vermont and New Hampshire are in a north-northeasterly direction. This coincides with the Connecticut River which probably follows an ancient drainage way.

Preglacial geology indicates extensive periods of erosion associated with the uplift of the Appalachian Mountains near the close

of the Paleozoic Era, and with other periods when the land was emergent. It is assumed that the present topography was well established prior to continental glaciation, including a well developed soil layer with superimposed streams including their meandering patterns.

Massive continental glaciation wore the topography into the currently existing subdued forms. Highlands were rounded on the upper side facing the glacier and steepened on the lower side away from the glacier. Stream valleys were eroded and smoothed, sometimes into the classic V-shaped glacial valleys.

The retreating ice redeposited morainal materials over the entire surface of the area. Stagnant ice blocks and frontal moraines created lakes that became sites of further deposition. While the lakes were in existence, material was deposited as sandy and gravelly terraces consisting of deep, well drained alluvial soils developing in medium textured sediments, derived mainly from schist, gneiss, granite, slate and phyllite. The type of alluvial material forming the flood plains adjacent to the Connecticut River comprise the Hadley soils as classified by the US Department of Agriculture, Soil Conservation Service, and are water-laid deposits composed mostly of silty, fine sands and nonplastic fine, sandy silts. Soil core samples were collected and analyzed. A bore hole log of the alluvial deposits is given on Plate 2.

c. Locality, Development and Occupation. The Northfield, Massachusetts site is located in the valley of the mid-Connecticut River Basin, noted for its rich alluvial soil which provides the foundation for one of the most productive farming areas in New England. Crops produced in the region include corn, tobacco, hay and assorted vegetables. The erosion site borders a large field used primarily for corn.

Vegetation in the areas not in agriculture is primarily woodland, including red and silver maple, elm, willow, cherry, poplar and alder. In the valley region, woodland is usually found on steep slopes and along the river. Some timber harvesting is done for lumber and firewood.

Northfield is a small agricultural, residential, and year-round resort community. Most of the labor force is engaged in construction, manufacturing, and the service industry. The population of Northfield (2,600 in 1970) has shown a slight steady increase since 1950. The town is served by 2 railroads and 2 major State roads, Routes 10 and 163.

d. Hydrologic Characteristics.

(1) Climatology. The relatively high elevations of the Green Mountains of Vermont and the Berkshire Mountains of Massachusetts influence the temperature, precipitation and snow cover of the central Connecticut River Basin which lies in the path of the prevailing westerlies and air masses moving predominantly from the interior of North America. Generally west to southwest air flow brings the hot dry weather which is responsible for occasional summer droughts. In the winter months, high pressure weather systems from Canada bring frigid air into the basin. Precipitation is moderate to heavy and well distributed throughout the year. The annual mean temperature is about 45°F.

Three general types of storms produce precipitation over the basin: continental, coastal, and thunderstorms. Continental storms originate over the western and central portion of the United States and move generally in an easterly or northeasterly direction.

Tropical hurricanes, the most severe of the coastal storms originate in the South Atlantic or Caribbean Sea. They usually move in a westerly direction then northerly and may be deflected by high pressure zones to New England. Hurricanes have occurred in the summer and fall months. Extratropical storms generally originate or intensify near the middle Atlantic States, travel northward along the coastline and generally occur in autumn, winter and spring.

The third type of storm is the thunderstorm which can be produced by local convective activity during the warm humid days of the summer months or be associated with a frontal system moving across the basin.

The average annual precipitation ranges from 43 inches in the main river valley to about 50 inches in the higher Berkshire and Green Mountains. Precipitation in the central portion of the basin during

the winter months is practically all in the form of snow. The average snowfall ranges from 50 inches in the valley to 70 inches in the mountains.

(2) Streamflow. Flow conditions in the Connecticut River in the area of concern are best represented by the records of the U.S. Geological Survey gage at Turners Falls Dam (D.A. = 7163 square miles) located at river mile 122.2 about 11 miles downstream from the Northfield site. About 50 percent of the 23-inch annual runoff occurs during the months of March, April and May. During this period combined snowmelt and rainfall create an especially great chance of flooding. The average annual flow at the gage is 11,890 cfs for the period of record (1915 to present). Average summertime (July-October) flow rate is 5,700 cfs and the mean spring flow rate (April-May) is about 31,500 cfs. The average annual peak discharge has been 71,500 cfs since 1961, the year of final construction of all major upstream storage reservoirs. A peak discharge of 210,000 cfs was experienced in the great rainfall-snowmelt flood of March 1936. The recurrence interval of this peak flow rate is estimated to be slightly more than 100 years. Operation of the system of 9 flood control reservoirs, constructed by the New England Division on upstream tributaries since 1936, would cause this flow rate to be a more rare event today. Minimum flows at the gage approach 0 cfs during periods of "nongeneration" at Turners Falls Dam. A minimum daily flow rate of 99 cfs was recorded in October 1944 at the USGS gage on the Connecticut River at Vernon, Vermont (D.A. = 6266 square miles) located about 10 miles upstream from the Northfield site.

e. Channel Conditions. Under normal to moderate flow conditions the Connecticut River passes through the well defined channel it has cut through the valley floor alluvium (Plate 1 and Photo 1). The river takes a meandering course and, typically, is continually eroding its banks which are generally steep and sloughing and consist mainly of silty sands and sandy silts. Overtopping of the riverbank occurs at a flow rate of about 90,000 cfs, or on the average only every 10 or so years.

Water surface levels under normal conditions vary widely on a daily

and weekly basis in the Turners Falls Pool. This reach of the Connecticut River is bounded by Turners Falls Dam on the downstream end and Vernon Dam on the upstream end. The flow regime is affected by the operation of both of these hydropower projects and by the Northfield Mountain Pump Storage Facility located mid-reach in the Turners Falls Pool and which uses the pool for both forebay (pump cycle) and afterbay (generation) during its operation. Water levels at Turners Falls Dam fluctuate about 3.5 feet daily, on the average and range between elevations 175 and 185 NGVD, the operational zone, on a weekly basis. Plate 5 shows a graph of typical weekly pool fluctuations.

f. Environmental Considerations. The bank stabilization project is expected to result in a net improvement in wildlife habitat and water quality. The riverbank origianlly was a severely eroded cliff of sloughing vegetation, mostly grass and shrubs, with a few large trees at the top of the bank. The stabilized slope, revegetated above toe protection structures with unmown grass, legumes, vines and shrubs, will provide an improved habitat corridor along the river. Bank stabilization and toe protection will also reduce siltation and localized turbidity in the Connecticut River. Removal of scattered trees along the top of the bank, and replacement with uniform vegetation and manmade structures on the lower slope will give the project an unnatural character in contrast to adjacent riverbanks. This effect will be minimized in the long term by natural revegetation with a wider variety of native plant species.

6. Demonstration Site - Test Reach.

a. Hydrologic Characteristics. The hydrologic aspects of the test site are as previously described in Section 5d. Hydrographs of average monthly and average daily discharges measured at Turners Falls Dam are shown on Plates 3 and 4, respectively. Ice usually forms along the shore around late December (actual time of occurrence varies from year to year depending upon coldness of air temperature) and often forms an entire cover on the river. The ice usually breaks up during the spring snowmelt runoff period in April.

b. Hydraulic Characteristics. As previously discussed in Paragraph

5e, the demonstration site is situated mid-reach in the Turners Falls Dam pool and is subject to regular daily and weekly water level and flow fluctuations (Plate 5) due to both downstream and upstream hydropower operations and to operation of a pump, storage facility located on the left bank a few miles downstream. An approximate stage vs. discharge rating curve for the Connecticut River at the Northfield site is given on Plate 6. Stream velocities along the bank were observed to be 2 to 3 fps during a discharge of 50,000 cfs. Under high floodflow conditions, hydraulic control of this reach of river shifts from the backwater effect of Turners Falls Dam to the natural control at the valley constriction located near the French King Bridge (river mile 126.3) and the water surface slope increases from the normal 0.1 foot per mile to about 0.3 foot per mile. Velocities near the bank under these flow conditons are not known.

Wind generated waves are a relatively insignificant cause of bank erosion at the demonstration site due to extremely limited fetch. Also, boat generated waves play a minor role compared to other causative hydraulic factors as there is no commercial navigation and only a small amount of recreational boating traffic.

c. Riverbank Description.

(1) Bank Materials. Materials composing the banks and valley floor of the Connecticut River are classified as silty fine sands and fine, sandy silt. Alluvial deposits in the vicinity of Northfield, Massachusetts are comprised mainly of the Hadley soils. Analysis of the boring log for the project site indicates a 1 to 2 foot layer of clayey sandy silt (ML) overlying a deep stratum of silty fine sand (SM). Soil classification is given on the boring log on Plate 2.

(2) Description of Vegetation. Vegetation in the test reach was primarily a narrow band of grass, shrubs and a few large trees between the river and a large agricultural field. The trees were primarily elm, red maple and cherry species. Shrubs included alder, dogwood, wild rose and other brambles, and sumac. Grasses and forbes were a typical mix of native and introduced species usually associated with agricultural areas in the Northeast.

(3) Erosion Conditions. Thirteen sites along the banks of the Connecticut River in the pool behind Turners Falls Dam (river miles 122 to 142) are eroding. Although rates of erosion have not been determined at the other locations, the Northfield site is eroding at about 1 to 3 feet per year. Most of the sites are subject to the sloughing type of erosion; a lesser number are subject to mass wasting, headcutting and shallow washing. The principal cause is shear stress associated with high streamflow. Other causes such as pool fluctuation, boat waves, overbank drainage, and seepage play lesser roles in the overall erosion process. Erosion prior to construction of the demonstration project is shown in Photo 1.

Northeast Utilities Company (NU) made extensive efforts to stabilize major segments of the banks of Turners Falls Pool, mainly through the use of tree clearing and hydroseeding during the mid-1970's. Some riprap revetment was also used. At the demonstration site the trees had been cleared and the raw bank had been hydroseeded. A complete description of the stabilization work by NU is given in Section 32 Program Inspection Report 6, by Malcolm P. Keown, Hydraulics Laboratory, U.S. Army Waterways Experiment Station, May 1979.

III. DESIGN AND CONSTRUCTION

7. General. Three different and somewhat innovative methods of streambank protection were used in the Northfield demonstration project. These included three types of revetment - precast cellular concrete block mattress, used auto tire wall and used auto tire mattress. All three revetment panels included vegetative protection on the upper bank. The arrangement of the various test panels is shown on Plate 7.

8. Basis for Design.

a. Lower Bank Protection. The primary goal in selecting the types of protection to be utilized was to gain experience with new and innovative methods of streambank erosion control. Precast cellular concrete block mattress was selected because of its commercial availability and

and the New England Division's desire to gain field experience with it. Used auto tire wall and mattress were selected on the basis that they are readily available materials and on the premise that they would require relatively simple construction techniques that local government agencies and private land owners could employ.

b. Upper Bank Protection. Upper bank protection was provided by a series of test areas of various mixes of grasses, legumes, vines and shrubs. In general, vegetation was used as an alternative to more expensive structural measures in the portion of the bank above normal high water line, and also as a more natural appearing bank cover in the project's rural setting. Selection of plant species was based on knowledge of suitable native and adapted species types commercially available in the region. Additional technical assistance was provided by the Soil Conservation Service (SCS), U.S. Department of Agriculture. Two methods of mulching were also selected for comparision: hay with plastic netting, and tobacco netting with wood fiber mulch. Grasses were selected to provide a thick vegetative mat that protects the soil surface from the erosive effects of rainfall and high stage river flows, and to buffer the impact of floating debris, induce minor silt deposition, and reinforce and stabilize the soil surface through extensive fine textured roots. Shrubs and vines, planted as container grown, healthy, young plants provide less rapid soil protection but better wildlife cover and food potential.

Plates 7 and 8 outline the plant species used and their location in the project. In general, the experimental grass mixes were chosen on the basis of rapid establishment, suitability for growth with little or no maintenance, and adaptability to periodic inundation. Legumes were added to some mixes for supplemental nitrogen. Shrub species were chosen for wildlife value and adaptability to periodic inundation. Vines were chosen for their hardiness, woody perennial growth and ability to provide rapid low cover.

9. Construction Details. The total project length is about 2000 feet. Three techniques of bank protection, described below and shown on Plates 7 and 9, were installed in reaches varying in length from 600 to 750 feet

each along the bank.

a. Reach 1. A precast cellular concrete block mattress (Nicolon Class 64 Gobimat) was constructed along the lower portion of the bank for a length of 600 feet (Photo 4). The mattress was formed by placing 4-foot wide by 20-foot long cellular concrete block mats at right angles to the river on a 2H to 1V graded slope above the normal low water line and on the existing slope below the normal low water line. The cellular concrete mats were plant assembled prior to delivery and placement, (Photo 2) and consisted of a series of concrete blocks, each measuring 8 inches wide by 8 inches long by 5 inches high and weighing about 15 pounds, glued to a plastic filter/carrier fabric. Voids in the cellular concrete mattress were filled with pea stone (Photo 3) and the mattress was anchored to the bank using No. 6 reinforcing steel rods spaced at 4 foot intervals along the top of the mats. The upper bank was formed to a 2H to 1V slope and seeded.

b. Reaches 2 and 3. An 18-inch thick rock berm was placed along the toe of the bank from the normal low water line to a depth of about 5 feet. A wall consisting of used auto tires was constructed above the rock toe protection on a 1H to 1V slope and to height of 7 feet above the rock toe (Photo 7). The auto tire wall was assembled in layers with each layer of tires being banded together with stainless steel bands (Photo 5). The auto tires were filled with crushed stone (Photo 6) and anchored to the slope with No. 6 reinforcing steel rods at each tire in the top row. Filter fabric (Nicolon 70/20 - woven) was placed behind the tires along Reach 2 for a length of 350 feet. Reach 3, identical to Reach 2 in all other respects had no fabric. The upper bank of both reaches were formed to a slope of 2H to 1V and seeded.

c. Reaches 4 and 5. The final section consists of an underwater rock blanket below the normal low water line and an auto tire mattress consisting of two layers of tires banded together with stainless bands and placed on a 2H to 1V slope above the normal low water line. The used auto tires were filled with crushed stone. Filter fabric (Nicolon 70/20 - woven) was placed under the mattress for the 350-foot length of Reach 4. Reach 5, also 350 feet in length, was not provided with fabric. The

mattresses were anchored to the slope with No. 6 reinforcing steel rods at each tire along the top. The upper bank was graded to a 2H to 1V slope and seeded.

10. Costs. Total cost and unit cost on a linear foot basis for each of the test reaches in the Northfield project are shown in the following table. These costs were developed from contractor bid items and do not include 6 and 7 percent markups for Corps of Engineers engineering and design, and supervision and administration, respectively.

Test Reach	Total Cost (Dollars)	Cost Per Linear Foot (Dollars)
1	119,700	200
2	54,500	145
3	41,000	109
4	62,150	178
5	48,900	163

Costs are based on 1980 prices. The final total contract cost for the project, which was completed in November 1981, was $411,634.

IV. PERFORMANCE OF PROTECTION

11. Monitoring Program. Due to the exhaustion of monitoring funds under the Section 32 program, no formal monitoring plan has been developed for the Northfield site. It is hoped that funds under some other authority will be made available to at least carry out a visual inspection program now that the project is completed. It is imperative to continue to observe this site in the future if any knowledge is to be gained regarding performance of the protective measures.

12. Evaluation of Protection Performance. As of this writing, construction of the demonstration project was just completed and thus no evaluation of performance of the erosion control measures is possible.

13. Rehabilitation. None required to date.

14. Conclusions. Since construction was not completed until November

1981, conclusions regarding performance of protective measures cannot be made at this time, nor are any expected to be reached in the future without the availability of funds to monitor performance.

NORTHFIELD PROJECT
PRE-CONSTRUCTION

PHOTO 1
FLOW FROM LEFT TO RIGHT
NOVEMBER 1976

PHOTO 1

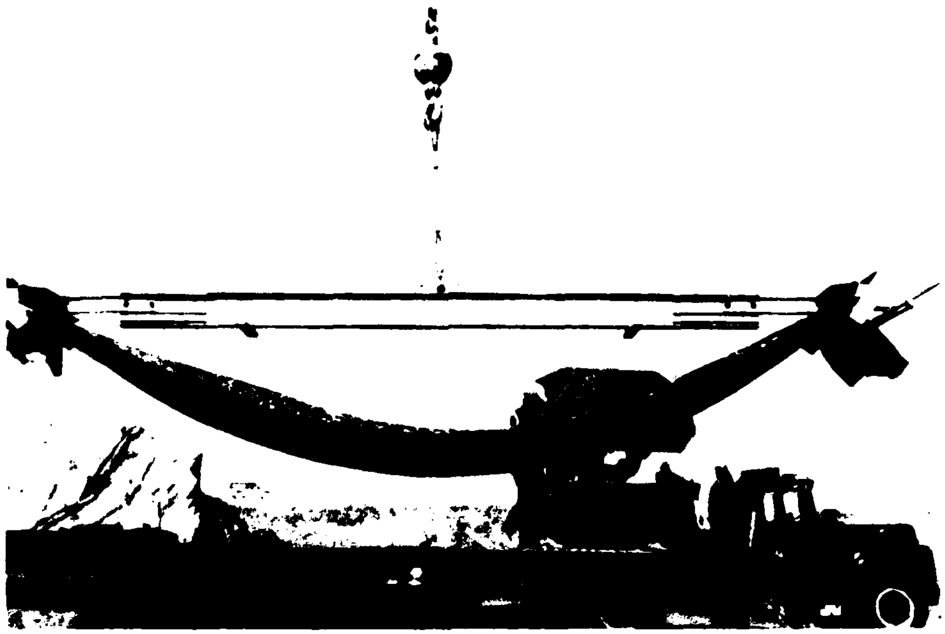


PHOTO 2
GOBIMAT
OCTOBER 1980

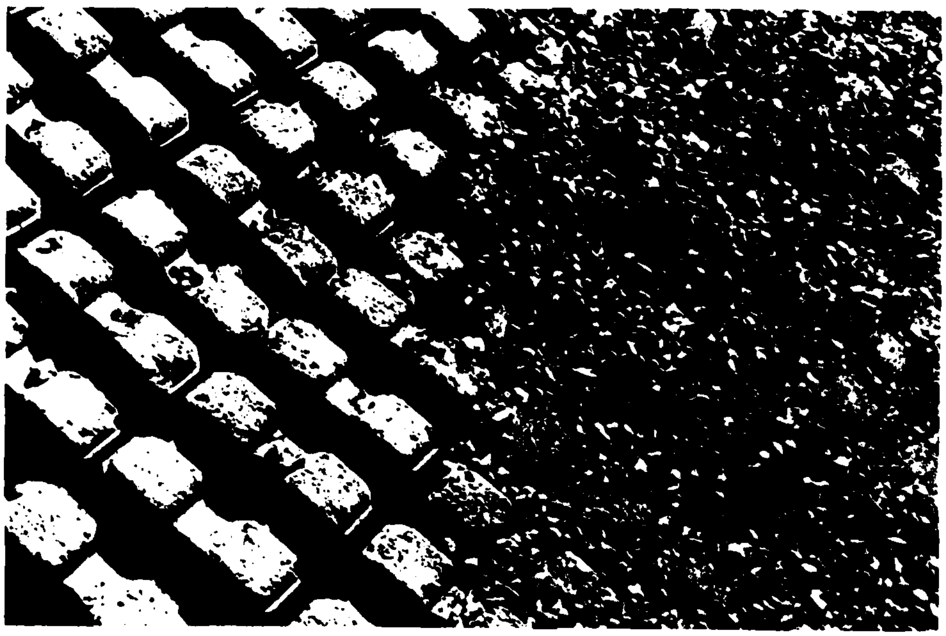


PHOTO 3
GOBIMAT CLOSE-UP
OCTOBER 1980

NORTHFIELD PROJECT
DURING CONSTRUCTION

PHOTOS 2 AND 3

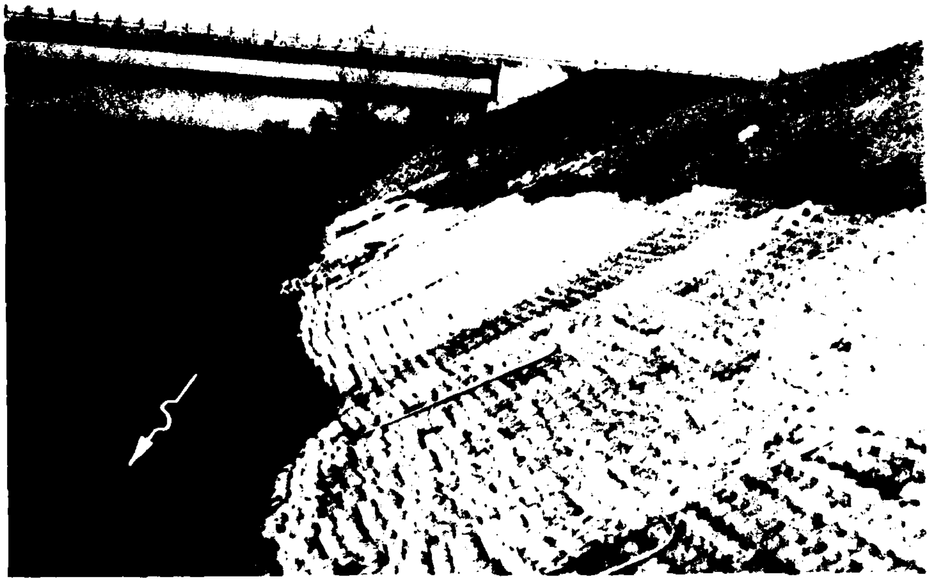


PHOTO 4
GOBIMAT
OCTOBER 1980

PHOTO 5
TIRE PREPARATION
OCTOBER 1980

NORTHFIELD PROJECT
DURING CONSTRUCTION

PHOTOS 4 AND 5

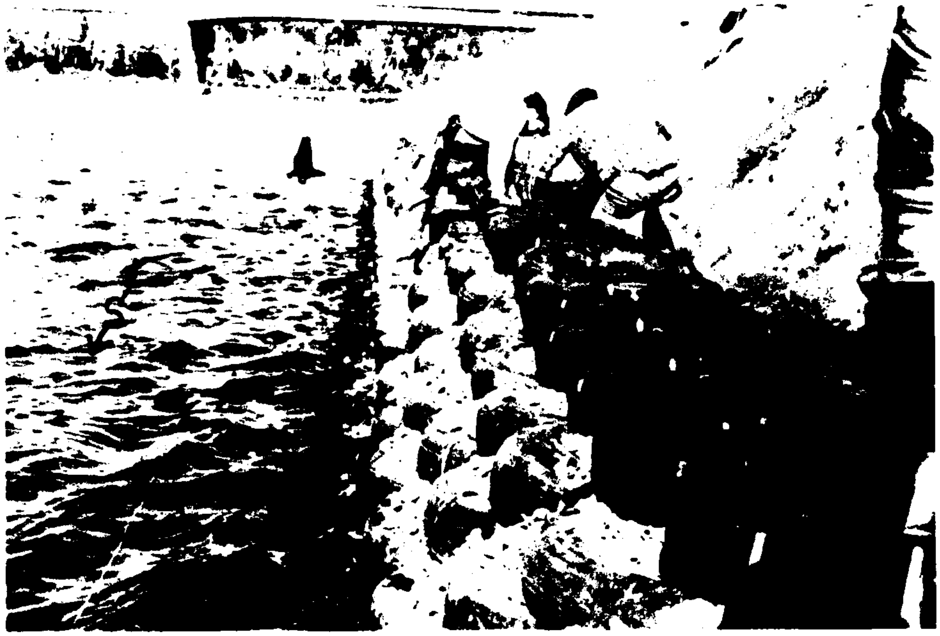
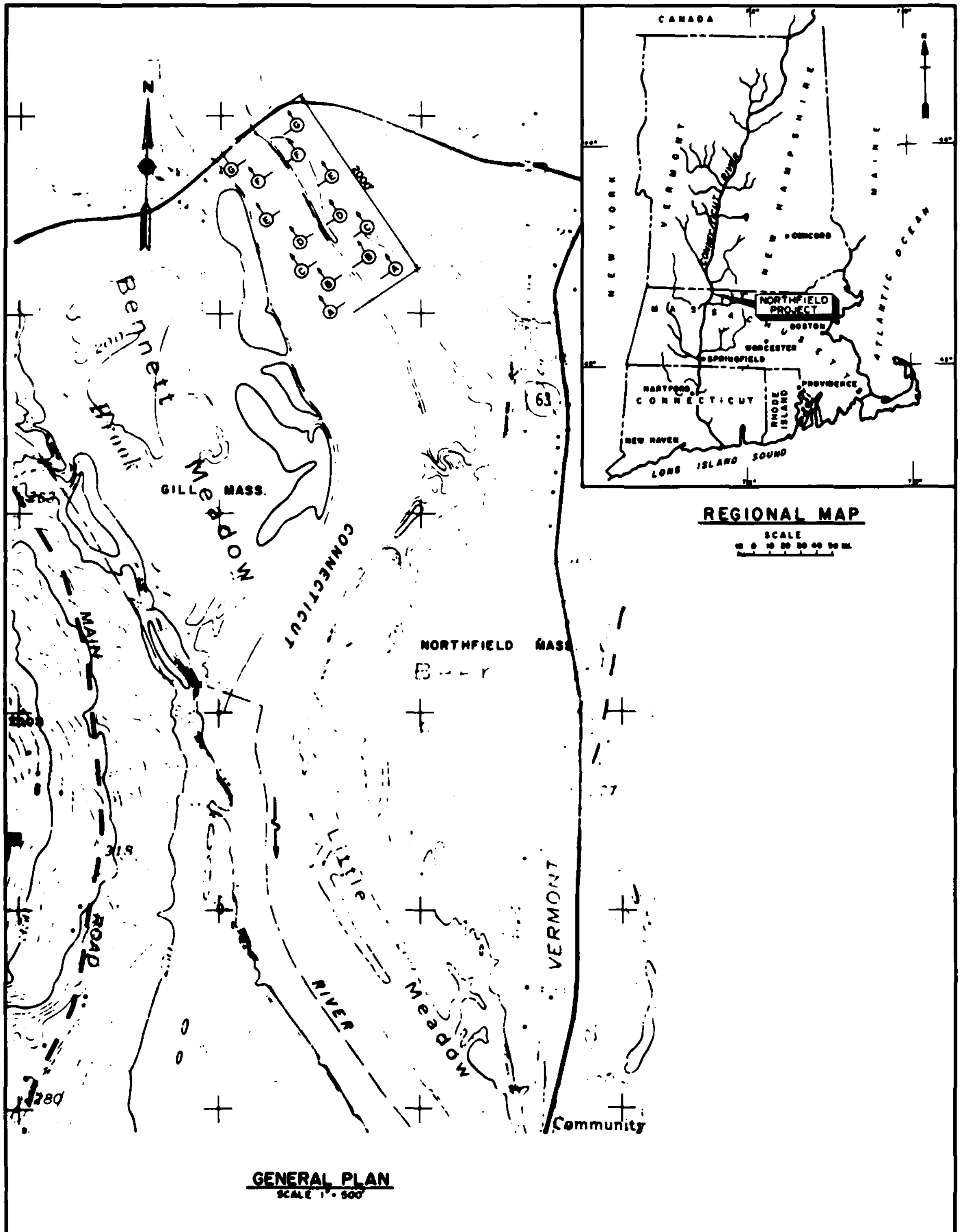


PHOTO 6
INSTALLING TIRE BULKHEAD
OCTOBER 1980

PHOTO 7
FINISHED TIRE BULKHEAD
OCTOBER 1980

NORTHFIELD PROJECT
DURING CONSTRUCTION

PHOTOS 6 AND 7



CONNECTICUT RIVER BASIN
NORTHFIELD, MASSACHUSETTS

PLATE 1

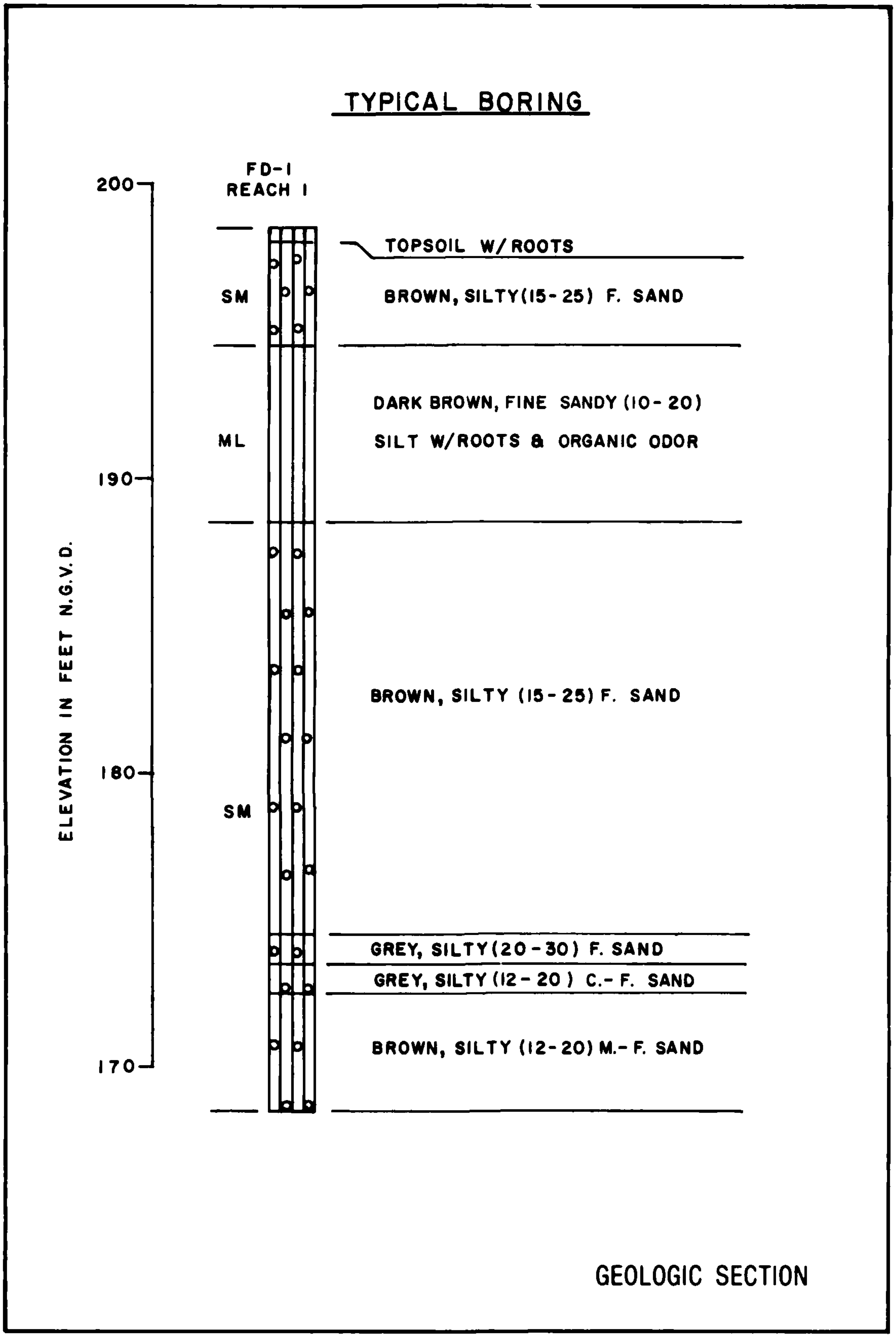

TYPICAL BORING
FD-1
REACH 1
200
190
180
170
ELEVATION IN FEET N.G.V.D.
TOPSOIL W/ROOTS
SM
BROWN, SILTY(15-25) F. SAND
ML
DARK BROWN, FINE SANDY (10-20)
SILT W/ROOTS & ORGANIC ODOR
SM
BROWN, SILTY (15-25) F. SAND
GREY, SILTY(20-30) F. SAND
GREY, SILTY (12-20) C.- F. SAND
BROWN, SILTY (12-20) M.- F. SAND
GEOLOGIC SECTION

PLATE 2

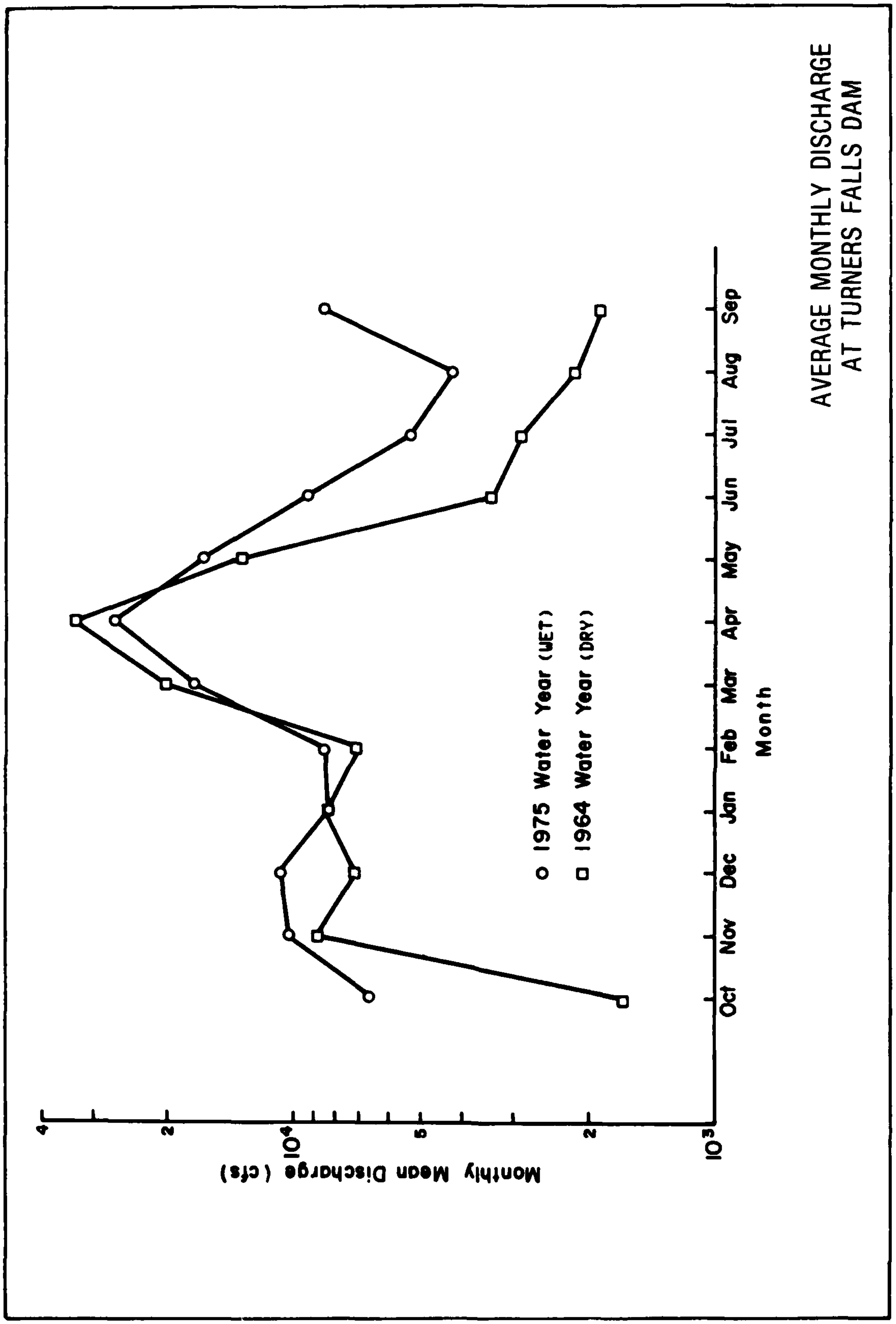

AVERAGE MONTHLY DISCHARGE
AT TURNERS FALLS DAM
1975 Water Year (WET)
1964 Water Year (DRY)
Monthly Mean Discharge (cfs)
Month
Oct
Nov
Dec
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
10^3
2
5
10^4
2
4

PLATE 3

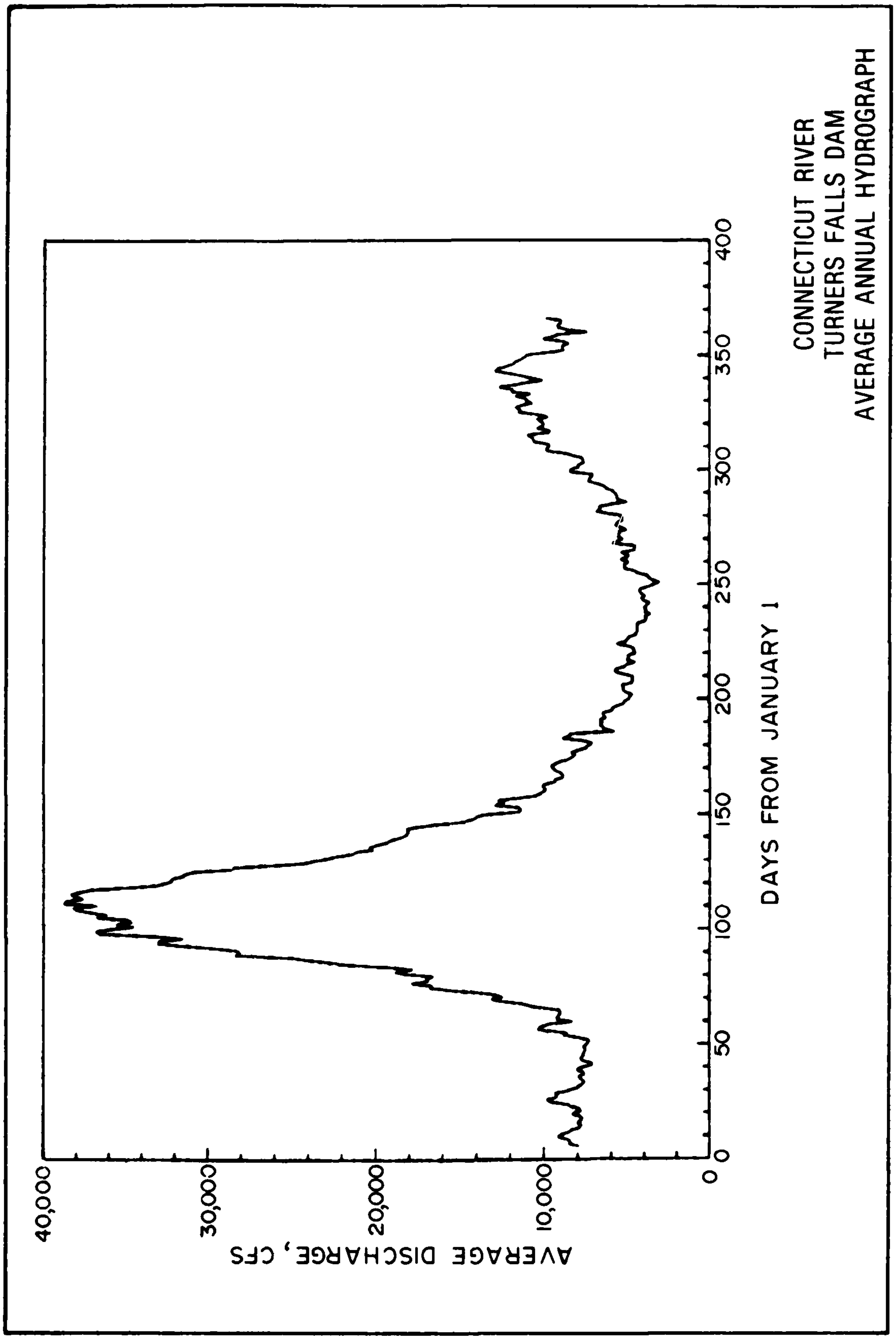
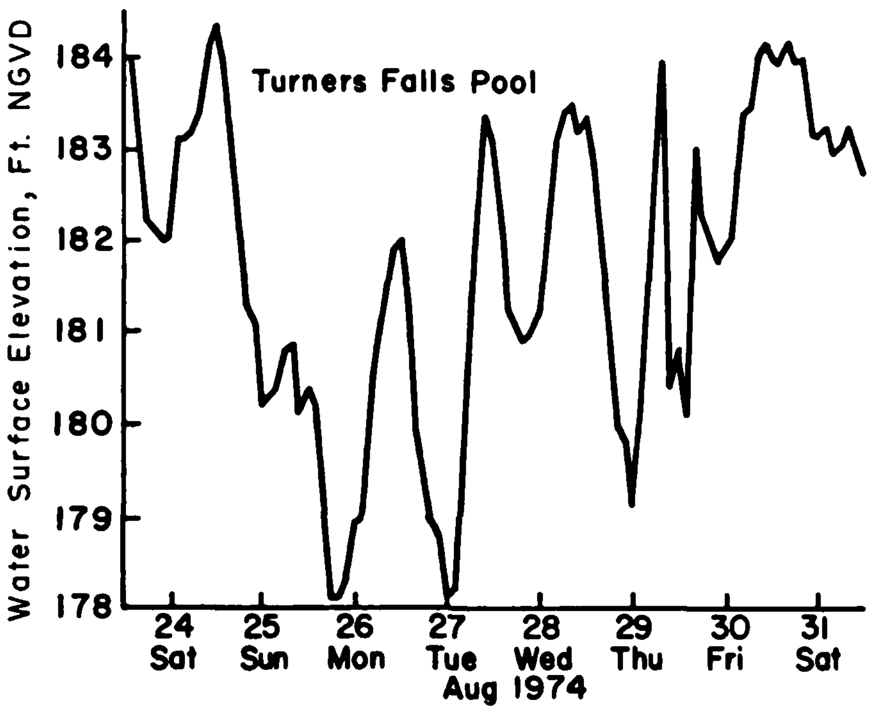
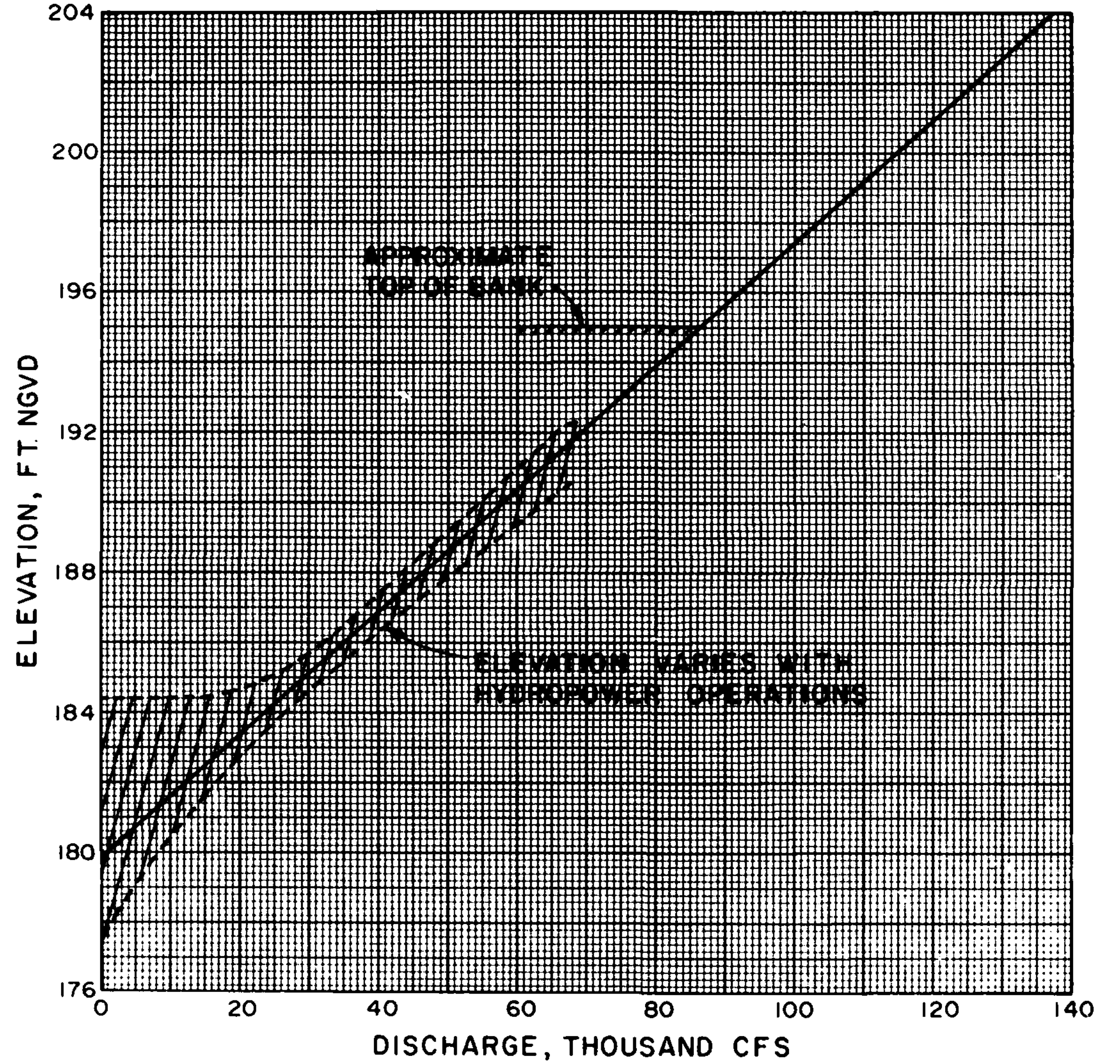

CONNECTICUT RIVER
TURNERS FALLS DAM
AVERAGE ANNUAL HYDROGRAPH
AVERAGE DISCHARGE, CFS
DAYS FROM JANUARY 1
0
10,000
20,000
30,000
40,000
0
50
100
150
200
250
300
350
400

PLATE 4



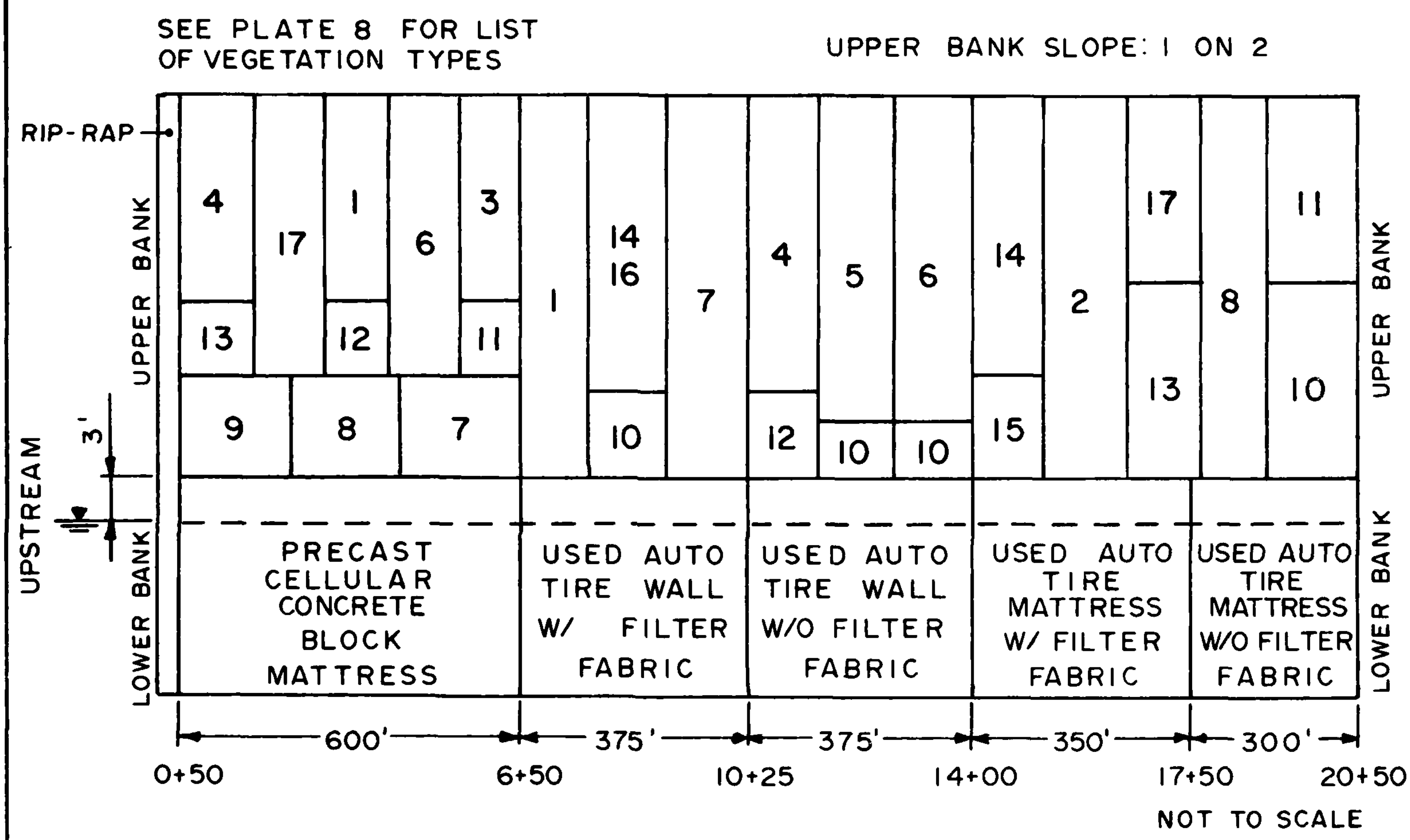
TURNERS FALLS
POOL FLUCTUATIONS

PLATE 5



APPROXIMATE STAGE
DISCHARGE RELATIONSHIP
NORTHFIELD DEMONSTRATION SITE

PLATE 6



CONNECTICUT RIVER
NORTHFIELD, MASSACHUSETTS
PROTECTIVE MEASURES

PLATE 7

SEED MIXTURES

1 REED CANARYGRASS
CREEPING RED FESCUE
REDTOP

2 REED CANARYGRASS
KENTUCKY 31 FESCUE
BIRDSFOOT TREFOIL

3 CROWNVETCH
KENTUCKY 31 FESCUE
CREEPING RED FESCUE

4 FLAT PEA
KENTUCKY 31 FESCUE
CREEPING RED FESCUE

5 CROWNVETCH
FLAT PEA
KENTUCKY 31 FESCUE

6 BIRDSFOOT TREFOIL
CREEPING RED FESCUE

7 KENTUCKY 31 FESCUE
CREEPING RED FESCUE
REED CANARYGRASS
REDTOP

8 KENTUCKY 31 FESCUE
CREEPING RED RESCUE
REDTOP

9 PERENNIAL RYEGRASS
REDTOP

SHRUBS AND VINES

10 PURPLE-OSIER WILLOW

11 SIBERIAN DOGWOOD

12 REDOSIER DOGWOOD

13 SUMMERSWEET

14 AMERICAN BITTERSWEET

15 VIRGINIA CREEPER

16 HALL'S HONEYSUCKLE

17 SWEETFERN

NOTE: NUMBERS ARE KEYED TO PANELS ON PLATE 7

NORTHFIELD DEMONSTRATION PROJECT
UPPER BANK VEGETATION

PLATE 8

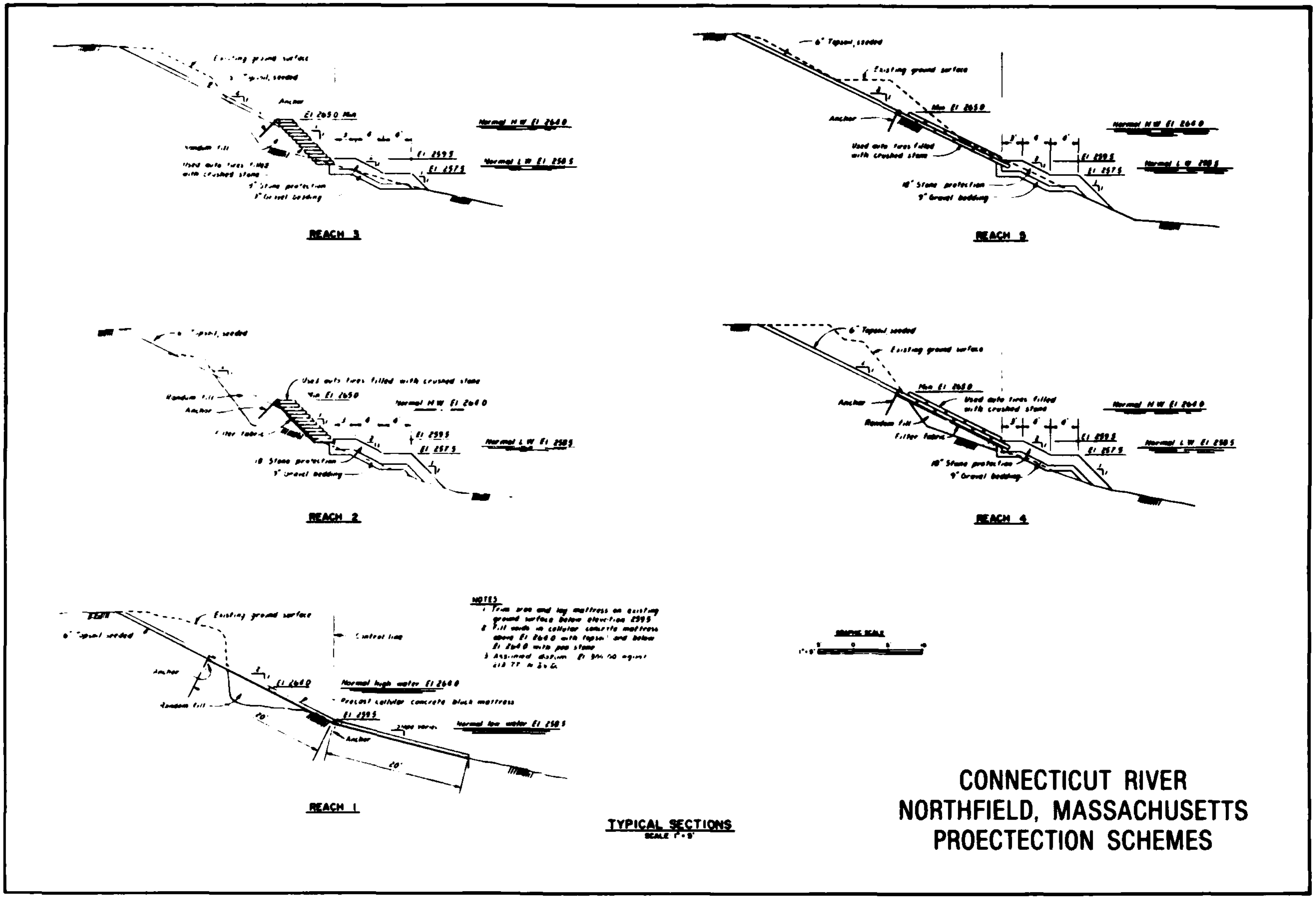

REACH 3
REACH 5
REACH 2
REACH 4
REACH 1
Existing ground surface
Anchor
Random fill
Filter fabric
Used auto tires filled with crushed stone
18" Stone protection
9" Gravel bedding
Normal H.W. El 264.0
Normal L.W. El 258.5
El 259.5
El 257.5
Min El 265.0
Normal high water El 264.0
Normal low water El 258.5
Precast cellular concrete block mattress
Control line
NOTES
TYPICAL SECTIONS
CONNECTICUT RIVER
NORTHFIELD, MASSACHUSETTS
PROECTECTION SCHEMES

PLATE 9

**DELAWARE RIVER AT
PAULSBORO, NEW JERSEY**

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

DELAWARE RIVER AT PAULSBORO, NEW JERSEY
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Paulsboro Protection Works, Delaware River - mile 89.7, Paulsboro, New Jersey. The location map is shown on Plate 1.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This Report describes a streambank erosion problem, the types of bank protection tested, and a performance evaluation of a demonstration project on the Delaware River at Paulsboro, New Jersey, constructed and monitored by the Philadelphia District.

4. Problem Resume. The problem existing at the site is erosion of the streambank. The area has experienced erosion at a rate of approximately 2 feet per year. The shoreline is characterized, generally, by a narrow beach area fronting a 20-30 foot high bank. Located along the top of the bank are various residential and commercial properties. Prior to implementation of the protective devices, the upper reach of the area was unprotected. The lower reach had some unprotected property and some

protected (bulkheaded) areas. However, some of these structures were in danger of undermining.

II HISTORICAL DESCRIPTION

5. Stream.

a. Geology. The site is located in the Coastal Plain Province which lies southeast of the Fall line. The crystalline basement rock of the Precambrian Wissahickon formation is overlain unconformably by gently dipping Cretaceous deposits which thicken from northwest to southeast. These deposits consist of alternating layers of pervious and impervious sediments and are overlain by younger Tertiary and Quaternary deposits which also consist of alternating layers of pervious and impervious sediments.

At the site the oldest unconsolidated sediments belong to the Raritan and Magothy formations of the Cretaceous Age. The Magothy formation is the younger of the two and consists predominantly of dark grey, commonly lignitic clay and layers of white fine sand. The Raritan formation is composed predominantly of clays that vary in color from white to red to gray and lesser thicknesses of light colored sands. The Raritan and Magothy formations at the site are mantled by the Pleistocene (Quaternary) Cape May deposits which range in thickness from 20 to 60 feet and consist of sands and gravels with occasional thin clay

layers. The Cape May materials were deposited as nearly continuous sheets during the interglacial period and prior to the last advance of the Wisconsin glaciation. During the retreat of the Wisconsin glacier these sediments were extensively eroded by the meltwater emerging from the ice. Some of the Cape May deposits now border Delaware River in a form of terraces.

The post glacial rise in the sea level submerged portions of the Coastal Plain and formed drowned river valleys, one of which is the Delaware Estuary. This submergence slackened the flow of Delaware River which in turn filled the valley with alluvium. In the vicinity of the site, these recent alluvial materials are within the Delaware River channel and consist predominantly of dark gray organic silts. Near the shore, the bottom materials consist of medium to coarse sands which are being derived by erosion from the bank.

b. Hydrologic Characteristics. The project area, located in the midlatitudes of the eastern region of the country, is primarily influenced by the movement of migratory weather systems from the west.

Air masses that influence the Delaware Valley are generally maritime tropical in summer and continental polar in winter. The air masses change frequently in the spring and fall, but they are either the very warm, moist masses from the Atlantic Ocean or Gulf of Mexico or the cold, dry air masses from Canada or the

midwestern United States. From time to time, short incursions of cool, damp maritime polar air from the North Atlantic cover the area, often as part of the flow around low-pressure areas moving northeastward up the Atlantic Coast, to produce northeasters. Mean annual temperatures in the area are 55-58 degrees F. Precipitation, about 44 inches annually, is well distributed throughout the year with more than 3 inches gen- erally reported almost every month. On the average the driest month is October with 2.5 to 3.0 inches.

c. Environmental Conditions. Water quality conditions within the estuary at Paulsboro were analyzed with respect to both the quality standards promulgated by the Delaware River Basin Commission (DRBC) and by the State of New Jersey. Fecal coliforms were far above the maximum geometric mean of 770/100 ml, with a mean frequency of 9152/100 ml calculated. The dissolved oxygen concentration mean during the summer months was 1.8 mg/l, below the DRBC minimum dissolved oxygen concentration of 3.5 mg/l for the summer months. Peak phenol concentrations (0.231 mg/l) exceeded the maximum permissible concentration of 0.02 mg/l. Peak concentration values for lead (0.121 mg/l) exceeded the maximum permissible concentration of 0.05 mg/l. In addition, oil is visible on the estuary waters, including that segment of the river in which an Oil refinery is located.

Benthic organisms such as turbifex worms, fingernail clams and certain midge larve occur in the project area. These species are particularly tolerant of organically polluted streams. For phytoplankton, green algae of the families Volvocales and Chlorocaccales were found more commonly during the summer and early fall than during the rest of the year. Algae of the phylum Euglenophyta were also more common during the warmest months. Rotifers dominated the zooplankton community. Of the seven genera of rotifers that were identified, *Brachionus* and *Kertella* were most abundant. Cyclopoid copepods and the cladoceran *Bosmina* were also found in a majority of the samples.

The Delaware River, in the vicinity of the project area, provides habitat for a variety of fish species. Year round residents include the common shiner, golden shiner, satinfin shiner, spotfin shiner, banded killifish, mummichog, silvery minnows and brown bullhead. The white perch, American shad, hickory shad, blueback herring, alewife and striped bass are common anadromous species of these waters. These species inhabit the waters of the Atlantic Ocean, entering the Delaware River during the spring months to spawn. The American eel is a catadromous fish species spending most of its life in fresh water (the Delaware River) but migrating to the sea to spawn.

The year round residents spawn predominantly during the late spring and early summer months. Most of these species spawn in

the tributaries of the Delaware River because the waters of the Delaware River are too swift and muddy and the substratum composed of sediments which are too fine. The banded killifish and mummichog prefer shallow waters and firm bottoms for spawning while the spotfin shiner prefers clean, gravelly streams.

The project area is highly urbanized with little open space. Two plots of cherry-sassafras forest, one of about 2 acres and another of about 1 acre, are the only forested land within the project area.

Cherry-sassafras typically occurs on recently disturbed sites. Black cherry, tree-of-heaven, and sassafras are the principal canopy species. Sweetgum, red maple, and various oaks are occasional associates. The undergrowth is composed of blackberry, sumac, Japanese honeysuckle, greenbrier, and wild rose.

Wooded and residential areas of the study area are expected to be utilized by numerous species of mammals including the Gray squirrel, cottontail, white-footed mice and house mice.

Species of birds expected to utilize the wooded lots of the project area include the mourning dove, common flicker, downy woodpecker, blue jay and Carolina chickadee. It is expected that the residential habitat would be utilized by species such as the rock dove, Chimney swift, house wren, house sparrow and shipping sparrow. Birds expected to utilize field and shrub areas include

the bobwhite, ringnecked pheasant, field sparrow, American goldfinch and song sparrow. Open water and shore habitat would be utilized by various species of waterfowl and by gulls.

6. Demonstration Site-Test Reach

a. Hydraulic Characteristics. The project area is situated in the tidal portion of the Delaware Estuary, which extends 135 miles from the mouth to Trenton, NJ. Its maximum depth is about 150 feet, while the average depth in the test reach is about 30 feet. The mean range of the tide varies from 4 to 7 feet throughout the estuary, averaging 5.7 feet at Paulsboro. The high tide at Trenton lags that at Cape May by eight hours. The tidal current at maximum ebb is between 2.0 and 2.5 knots.

The primary erosion mechanism appears to be wave action generated by passing vessels. The magnitude of these waves varies depending on vessel size, draft, speed and proximity to shore. The waves generated intersect the shoreline diagonally creating a spiraling, cork-screw effect which tears at the bank material with a peculiar churning action. The fine material in the beach is drawn out into deeper water as is indicated by the gravelly nature of the beach. The most significant erosion occurs during high tide or periods of abnormally high river levels. At high tide, the wave action attacks the base of the bank causing a subsequent slumping. Waves generally reaching the shore range from two to three feet in height.

Presently a remote wave monitoring program is being conducted at the test site by the Waterways Experiment Station (W.E.S.). The purpose of this program is to gather detailed data on the wave characteristics at the site. Information collected will consist of wave height and period. In conjunction with this effort ship traffic data will be collected and correlated with various wave parameters. This information will be collected and reduced to usable format and then forwarded as a supplement to this report.

b. Riverbank Description

(1) Bank Materials.

The high banks paralleling the river consist of loose, highly erodible brown fine to medium sands of the Cape May formation. There is no evidence of stratification apparent in the exposures of this formation at the site. The beach materials found at the north end of the project consist of reworked sands from the high banks.

(2) Normal Bank Vegetation. None. Except for a narrow upland fringe along the shoreline, the area is developed.

(3) Bank Erosion Tendency. The test site (Plate 1) had been eroding at a rate of 2 feet per year for the past several years. The rate of erosion is generally constant due to the major impact of ship wake wash and tide fluctuation. Paulsboro Sportsmen's Association installed a bulkhead to retain the

embankment along 270 feet of their property. That bulkhead has been reasonably successful; however, erosion along the toe threatened the stability of the bulkhead. The Association is currently planning to construct a new bulkhead at this location to replace the deteriorated structure.

In the area immediately upstream of the Sportsman Association bulkhead, the toe of the bank is protected somewhat by a low concrete retaining wall. This wall is fronted by a beach area that is eroding.

III. DESIGN AND CONSTRUCTION

7. General. Four types of revetment and two types of groins were constructed at the Paulsboro site. Scrap rubber tires, gabions, stone rip-rap and stone rip-rap with a wooden bulkhead were placed to test the revetment scheme. Separate reaches were tested with groins constructed using Longard Tubes and nylon sandbags. A location plan for the site is shown on Plate 2. Additional details including cross section, material quantities, labor estimates, costs and construction methods are summarized in the attached Appendix.

IV. PERFORMANCE OF PROTECTION

8. Monitoring Program. A monitoring program was established for the demonstration sites with the purpose of analyzing the

functional and structural performance of the individual erosion control devices. The intent was to develop the maximum amount of information on the erosion control devices and disseminate that information to landowners concerned with the ever-increasing problems of streambank erosion.

Monitoring guidelines were established by the Waterways Experiment Station. Because the demonstration site is located in the tidal portion of the Delaware River and the cause of the erosion appears to be wave action generated by passing vessels, the general guidelines needed site specific modification to best fulfill the purpose of the monitoring program.

The monitoring program involved three periods of data collection; the pre-, pending, and post-construction periods for the sites and installed devices. The quantitative information obtained for each site contained information on the base conditions, the structural aspects of the established devices, the post construction behavior of the test structures, and response of the shoreline.

The following items of data collection were included in the program:

a. <u>Topographic and Bathymetric Surveys</u>. Topographic survey data was obtained in order to establish:

(1) The functional behavior of the device by monitoring changes in the beach and nearshore bottom;

(2) The structural integrity of the device by monitoring the condition of the structures.

Surveys were taken before and after construction to document the pre and as-built conditions. Surveys were continued quarterly thereafter, as well as after storms that caused significant changes in the beach profile. Profile lines extended at least to 2 ft. below mean low water. Rods were installed at various points along the beach to readily observe trends in profile configurations.

b. Visual Inspection. Inspections of the sites were made monthly by technical representatives of the Philadelphia District. The various aspects of these sites are explained, generally below.

(1) The functional performance of the structure was observed by noting whether the structure caused or prevented erosion or caused accretion above or below the waterline, to the left or right of the structures.

(2) The structure was inspected for soundness to determine the cause and type of deterioration or to determine if repairs were required to restore the function of the structure. Repairs generally were not performed on a test device; however,

in some cases repairs were necessary to prevent adverse or biased effects.

(3) Surticial sediment samples were taken to establish pre- and post- construction conditions and quarterly, thereafter, to document the composition and changes in the beach. Standard gradation analyses were performed on the samples to determine the type of material being trapped by the structure.

c. Photographic Documentation. Ground level photographs were taken during the visual inspection to document the functional behavior and structural integrity. The items photographed were:

(1) The beach and bank to document the changes to the left and right of the structure;

(2) The structure to show its condition;

In addition to ground level photography, vertical aerial color photography was taken on a quarterly basis. The purpose of this photography was only to monitor any radical erosion and/or accretion trends at the site.

d. Groundwater Monitoring. Piezometers were installed behind the tire and gabion revetments. The purpose of the piezometers was to monitor the groundwater flow through the structure to assist in analyzing the performance ot the filter materials used. Data from the peizometers was collected during the visual inspection.

12. Evaluation of Protective Devices

a. Shoreline Changes. To date the structural modifications have apparently been successful in preventing erosion in the beach area. Erosion of the upper portion of the high banks at the northern end of the project caused by surface runoff is continuing. The products of this erosion are presently being deposited on the berm behind the rubber tire and rip-rap revetment section. This eroded material is not affecting the stability of the structures, however this upper bank erosion should not be overlooked in future project of similar nature.

b. Structural and Functional Performance (by structure).

1. Nylon Sandbag Groins. The nylon sandbag groins have performed satisfactorily in that they have reduced or practically eliminated erosion. In fact, some sand has actually accreted adjacent to the bags. However, as structural units they have performed poorly. They are subject to vandalism such as knife cutting and burning, and are susceptible to damage from elements such as ice, floating timber and floating debris. The only feasible solution to the vandalism and damage problems would be filling the bags with cement and sand mix, giving the bags the function of temporary forms only. If this cement-sand mix is used, the cost per linear foot of struture will increase considerably.

2. Used Tire Bulkhead (without toe protection). During the period of analysis the used tire bulkhead has retained and

prevented erosion of the embankment. There has been uneven settlement of individual tire stacks which has created some isolated gaps between adjacent stacks. This settlement has resulted in the loss of some of the gravel fill inside the individual units. However, with the implementation of the filter cloth backing, there has been no discernible loss of material from the backfilled area.

Due to the non-uniformity of size of the tires, specifically the width and diameter, there were problems maintaining a tight seal between adjacent units.

2a. Used Tire Bulkhead (with toe protection). Efficiency similar to portion of bulkhead without the stone toe. No appreciable difference in performance has been noted through the analysis period.

3. Riprap Revetment. The riprap revetment is performing very well, and has provided excellent bank erosion control. No problems have been experiencd with this structure and it is highly recommend for use in erosion control projects.

4. Longard Tube Groins. The longard tube groins have performed satisfactorily in reducing or eliminating erosion, by containing the sand beach. Moreover, a small accretion of sand actually formed adjacent to the groins. However, the structural

soundness of the system is impaired by vandalism. Tubes have been cut with knives and hatchets, and punctured with metal objects. Two types of protective coatings were installed: one with a sand-epoxy mixture cover, and the other with aluminum sheeting strapped around the tube. Both protective coatings proved to be very unreliable and unsatisfactory. The only feasible solution to the vandalism problem would be to fill the longard tube with cement and sand mix. However, using the cement-sand mix, the cost per linear foot of structure will increase considerably.

5. Toe Protection - Existing Bulkhead. The stone riprap, used as a toe protection for the existing and deteriorated wooden bulkhead, has maintained its structural integrity and has been performing satisfactorily in resisting erosion and scouring by waves.

6. Gabion Revetment. The gabion revetment system has performed satisfactorily, as designed. No natural deterioration or vandalism has been noted. This revetment appears to be an excellect method of bank stabilization.

13. Rehabilitation. The original plan for the scrap tire bulkhead provided for placement of a reinforcing rod through the top of each pile and a concrete cap. The purpose of this was to prevent uplift of the tires by vandalism and a potential loss of

material. Subsequent to the completion of the construction of this device and prior to completion of the project, the bulkhead experienced settlement of individual tire stacks. Since the top tire was fixed in place by the reinforcing bar and concrete cap substantial gaps appeared just below the top tires.

In order to alleviate this problem the structure was modified by removing the bar and breaking the concrete cap. This allowed for a uniform settlement of the individual stacks. In addition, to prevent uplift of the tires, the tie back cables were disconnected and replaced over the top tire.

14. Conclusions. All four types of revetment tested at the site have been effective. However, the scrap tire revetment structure may require more maintenance than the two stone rip-rap sections tested. Both the Longard Tube and nylon bag groin structures were effective in retaining beach sand at the Paulsboro site. Although both of these types of groins proved effective in protecting the embankment, vandalism committed on those structures caused extensive damage that would result in high costs for rehabilitation.

Reference is made to the upper bank erosion, noted in paragraph 12.a., behind the scrap tire bulkhead and rip-rap revetment. Although this erosion has had no negative effect on the structural stability of the two devices to date, this problem

should not be overlooked in future design of protective devices of a similar nature.

All four types of revetment tested at the site were effective in arresting the erosion cycle. Table 1, below, presents a relative ranking of the devices tested on the basis of per foot of bank protected. This assumes no vandalism occurs and does not include any projected maintenance.

At this site the most cost effective system was the longard tube groin system based on linear foot of beach protected for local community performance. The system requiring least equipment is the stone rip-rap revetment.

TABLE 1

RELATIVE COST COMPARISON

Project System	Contract* Cost	Structure Length	Unit Cost (per L.F.)	Bank Protected	Unit Cost (per L.F.)	Relative** Rank
1. Nylon Sand Bag Groin	$45,369	360 L.F.	$126	460 L.F.	$99	.64
2. Scrap Tire Bulkhead	$46,012	300 L.F.	$153	300 L.F.	$153	1.00
3. Stone Revetment	$38,419	250 L.F.	$154	250 L.F.	$154	1.00
4. Longard Tube Groins	$37,000	500 L.F.	$74	740 L.F.	$50	.32
5. Stone Toe Protection (Existing Bulkhead)	$24,391	270 L.F.	$90	270 L.F.	$90	.58
6. Gabion Revetment	$37,981	205 L.F.	$185	205 L.F.	$185	1.20

* Contract Cost represents structure in place, includes modifications.
** Relative Rank based on Stone Revetment, per foot of bank protected.

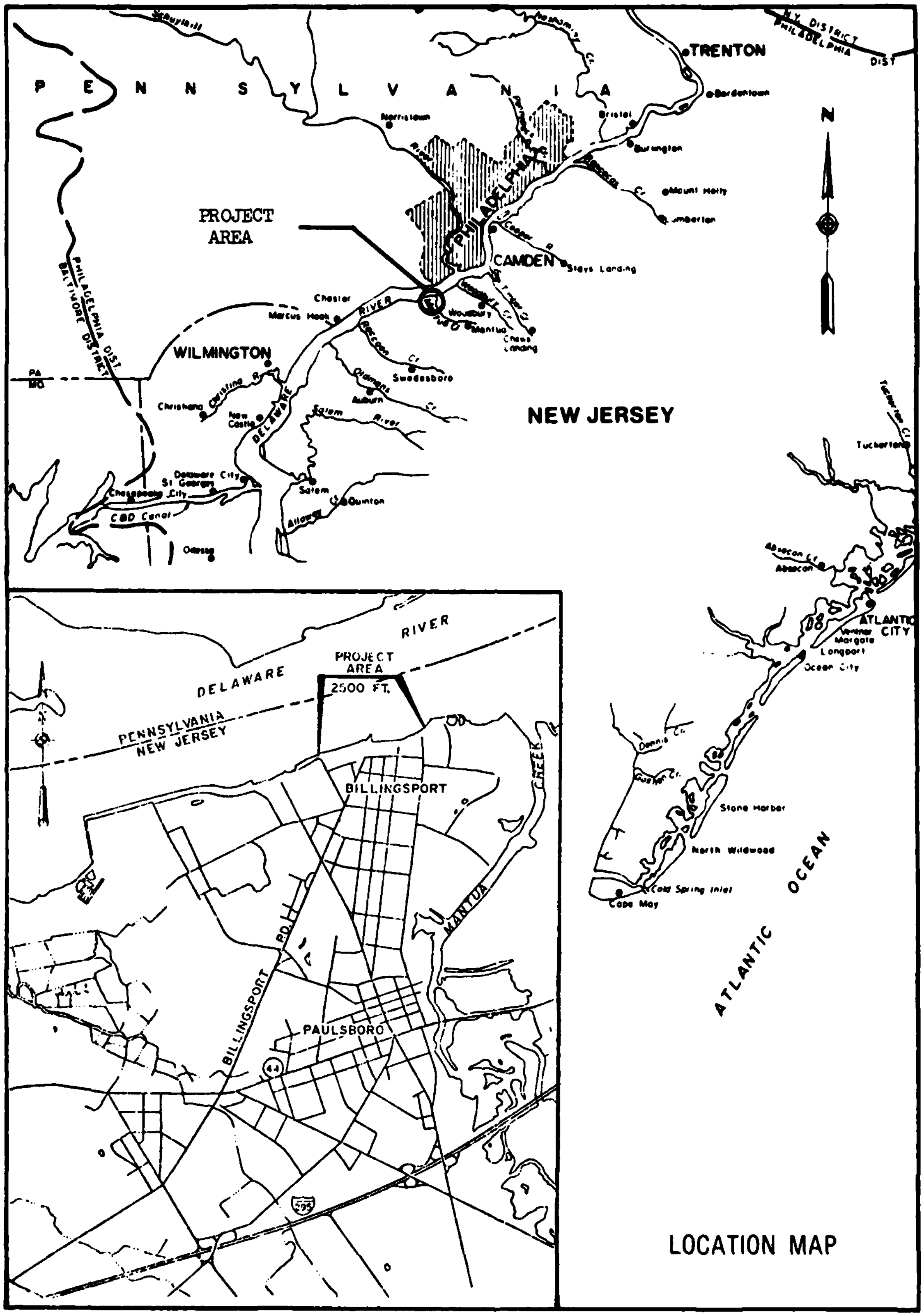
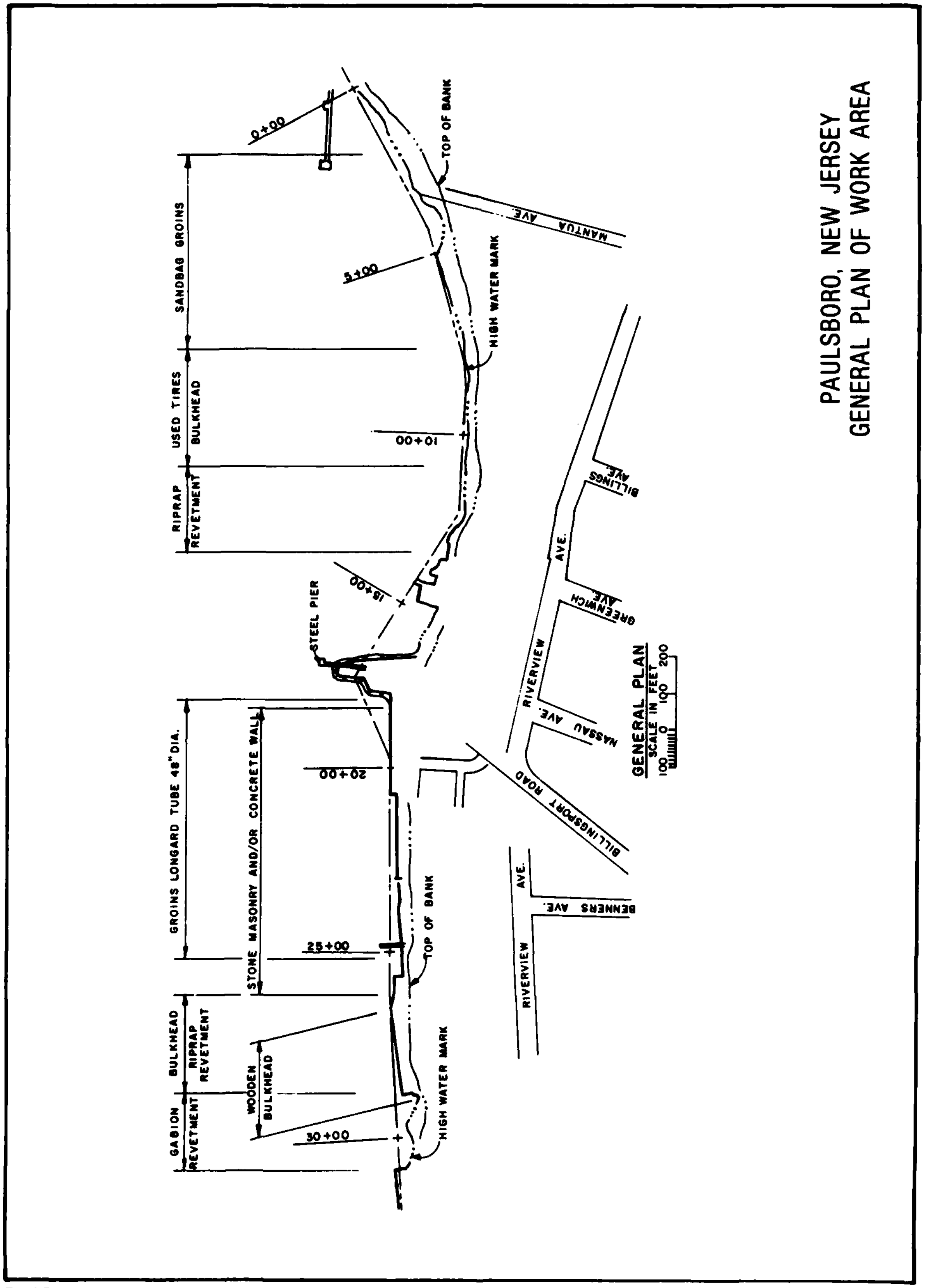

PENNSYLVANIA
TRENTON
N.Y. DISTRICT
PHILADELPHIA DIST
PHILADELPHIA
PROJECT AREA
CAMDEN
Chester
Marcus Hook
DELAWARE RIVER
WILMINGTON
PHILADELPHIA DIST.
BALTIMORE DISTRICT
PA
MD
NEW JERSEY
Salem
C&D Canal
ATLANTIC CITY
Ocean City
Stone Harbor
North Wildwood
Cape May
ATLANTIC OCEAN
N
PROJECT AREA
2500 FT.
DELAWARE RIVER
PENNSYLVANIA
NEW JERSEY
BILLINGSPORT
MANTUA CREEK
BILLINGSPORT RD.
PAULSBORO
44
295
LOCATION MAP

PLATE 1



PAULSBORO, NEW JERSEY
GENERAL PLAN OF WORK AREA

PLATE 2

APPENDIX

CONSTRUCTION STATISTICS
ON INDIVIDUAL DEVICES

SCRAP TIRE BULKHEAD

MATERIALS

Quantity:

173 salt treated wood piles - 8" x 10' long
1,100 used rubber tires - 14" - 15"
173 sections of #4 reinforced bars
86 lengths of 5/8" cable
14 c.y. of 2500 psi concrete
26" deadmen piles, 12" x 10' long
456 c.y. of granular material
filter cloth

EQUIPMENT

1 Bucyrus Erie 30B Crane, 35 Ton, w/3000 lb. drop hammer and lead
front-end loader
hand tools
air drill

LABOR

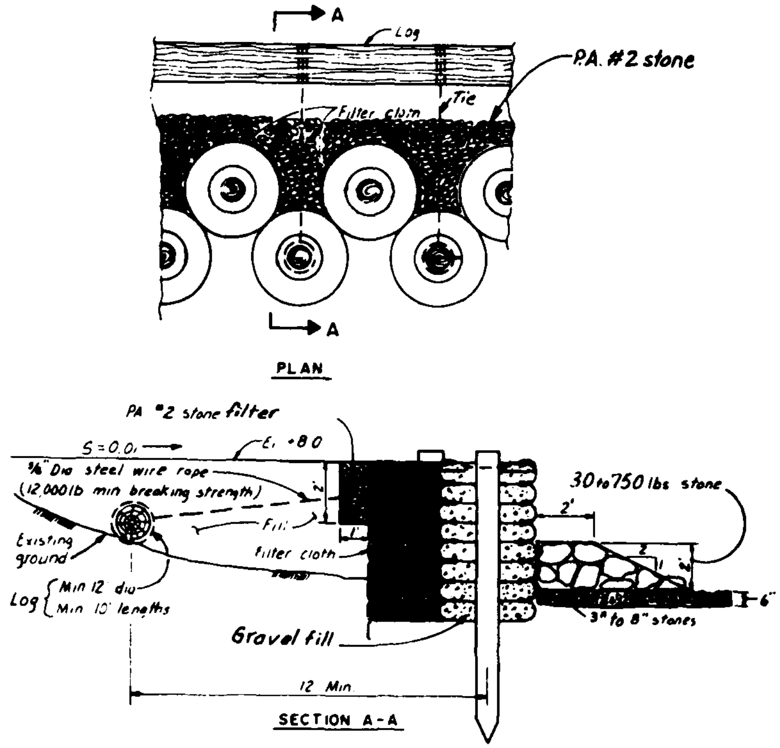
1 Superintendent
1 Operator
4 Laborers: 1000 MN. Hours

COST Contract Amount = $46,012.00
(300 ft.) - $153 per l.f of structure

METHOD & SEQUENCE OF CONSTRUCTION

Piles are driven at low water line parallel to the Delaware River. When all 173 piles were in place, the contractor excavated the equivalent of two tires deep around each pile in preparation for placement of tires below ground. The tires are hand packed with gravel and placed over piling. The top tire is reinforced with #4 rebar driven through each pile and filled with 4" thick, 2500 psi concrete. Deadmen are laid parallel, 12 foot behind the tires and attached by 5/8" cable to each outside row pile. Filter cloth is placed on the inside face of the tire bulkhead and the area behind the bulkhead is filled with a granular material. The filter cloth is to prevent fill from washing into the river through gaps.

SCRAP TIRE BULKHEAD



The scrap, rubber tire revetment is 300 feet in length with a top elevation of + 8.0 D.R.D. The structure consists of individual modules of tires arranged in a staggered alignment. Each module consists of tires mounted on a wood pile driven 6 feet (min.) into the ground. The top elevation of the piles are approximately +8.5 D.R.D. The tires are held in place with a reinforcing bar placed through the top of the pile and bent downward on the tires. Filter cloth is placed behind the structure to minimize the loss of backfill through the structure. In addition, a 2 foot deep by 1 foot wide gravel filter, of PA. #2 aggregate stone is installed immediately behind the structure to facilitate drainage.

In order to evaluate toe scour conditions at the bulkhead, half of the structure had toe protection placed. This consists of stone, 30# to 750#, placed on 6" of filter stone (3"-8") placed on filter cloth. The stone protection has a maximum height of 2 feet with a front slope of 1 on 2.

STONE RIPRAP REVETMENT

MATERIALS

Quantity: 10-750# lbs rip-rap
gravel apron, 12' wide (min.) 3" - 8" gravel
filter cloth
0-40#, mat stone

Supplier: Stone - Maryland Materials, Inc.

EQUIPMENT

1 gradall
1 front end loader

LABOR

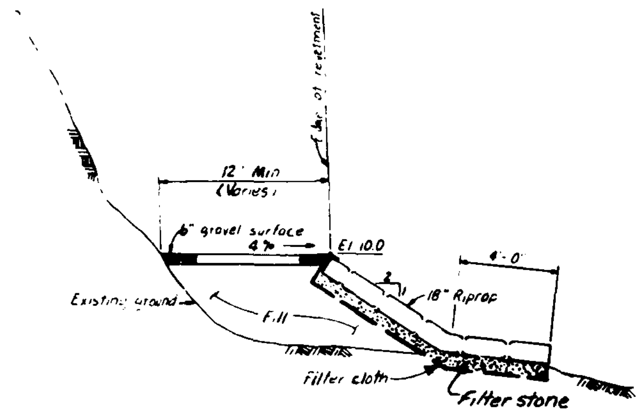
1 supt.
2 operators
2 laborers
Total Labor - 200 manhours

COST contract amount = $38,419
(250 ft) $154.00 per l.f. of structure

METHOD & SEQUENCE OF CONSTRUCTION

The compacted granular material is placed to the required elevation and using the gradall is cut to a 1 on 2 slope. The filter cloth is then placed and a 6" cushion of mat stone (1-40 lbs) is placed on the filter cloth. Using the gradall the riprap is laid, working from the bottom of the slope to the edge of the revetment.

STONE RIPRAP REVETMENT



The revetment approximately 250 feet in length, consists of 18 inches of riprap, 10# to 750# stone, placed on a 6" filter of 3"-8" stone on filter cloth. The slope of the revetment is 1 on 2. The structure has a 12' (min.) top width of 6 inches of compacted gravel with a 4% slope to facilitate drainage and prevent scour from runoff. The top elevation of the structure is approximately +10.0 D.R.D.

LONGARD TUBE GROINS*

MATERIALS

Quantity:
- -4 Longard tubes (40"dia)
- -4 Longard tubes (10"dia)*
- -bar sand
- -filter cloth
- -epoxy covering
- -.032 thick aluminum sheeting

Description: -2-ply flexible high density Polyethelene Longard tubes

Fill material - bar sand

* groins #5 and #6 have a filter cloth base with 10" tubes sewn on each side of the cloth to hold it in place.

EQUIPMENT

1-Caterpillar 977H, 3 1/2 C.Y. Front End Loader
1-sand hopper
1-40 hp water dump
hand tools

LABOR

1-Supt Dockbuilder
1-Equipment Operator
1-Dockbuilder
1-Laborer
Total Labor - 206 manhours

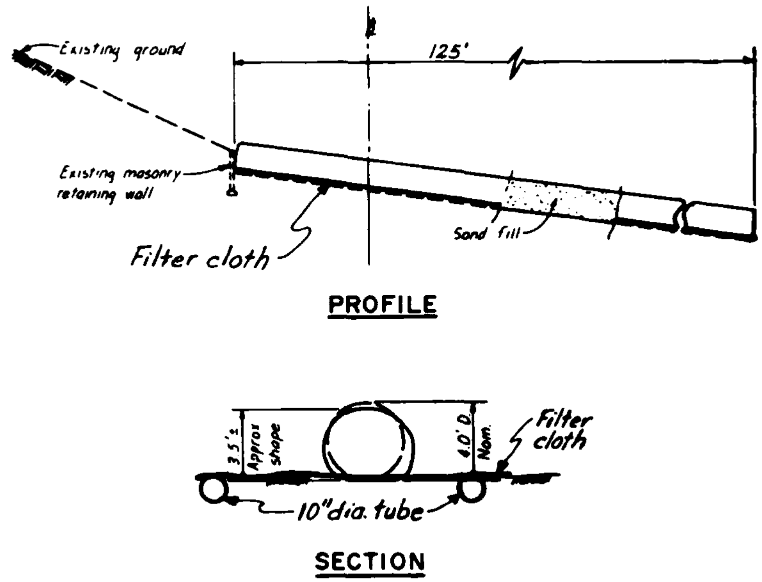
COST contract amount = $37,000
(500 ft.) - $74 per ft. of structure

METHOD & SEQUENCE OF CONSTRUCTION

The location of the Longard groin was staked out. The head section of the tube was placed against the existing seawall and hopper connections were made. Water was pumped into the tube to expand it to its natural shape. Following the water, the bar sand was pumped into the tube until completely filled.

*This is a patented system with the patent granted on the device for placing sand fill.

LONGARD TUBE GROINS



The four groins are 125 ft. long, 40" Ø tubes, composed of 2-ply flexible high density Polyethelene. Each groin is placed on the existing grade and spaced at 200 ft. intervals. In order to analyze the settlement, a 10 ft. wide woven filter cloth is placed under 2 of the groins. The filter cloth is anchored on each side by a 10" Ø tube sewn to the edges of the cloth.

Since the tubes may be subject to vandalism, 2 types of protective coatings were tested; a sand-epoxy mixture and .032" thick aluminum sheeting. The sand epoxy mixture is painted on the exposed portion of the tubes while the aluminum sheeting is strapped around the tube.

BULKHEAD TOE PROTECTION

MATERIALS

-10# - 750# riprap stone
-270 l.ft. of filter cloth

EQUIPMENT

1-Komatsu D775-3, 2.9 C.Y., Front End Loader
hand tools

LABOR

1-Superintendent
1-Operator
2-Laborers
Total Manhours - 64.0

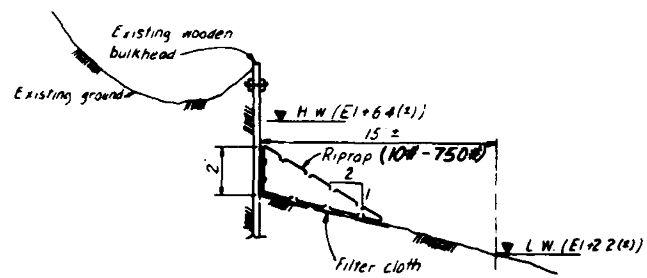
COST

Contract Amount - $24,391.50
(270) - $90 per foot of structure

METHOD AND SEQUENCE OF CONSTRUCTION

The filter cloth was placed along the slope of the existing bulkhead. The riprap was then placed on the cloth.

BULKHEAD TOE PROTECTION



This test section consists of approximately 270 feet of toe protection for an existing bulkhead. The structure is comprised of 10# - 750# riprap placed on filter cloth on a 1 on 2 slope. The stone is 2 ft. thick at the bulkhead.

GABION REVETMENT

MATERIALS

-84 - 3' x 6' x 1.5' gabion baskets
-4" to 8" stone
-160 cy backfill
-rip-rap

EQUIPMENT

1-Bantaam gradall
1-small dump truck
1-generator/lights
Hand Tools

LABOR

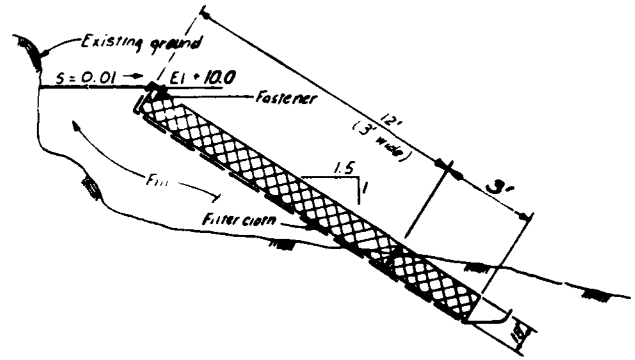
1 Superintendent
1 Operator
2 Laborers
Total Manhours - 414.0

COST Contract amount - $37,981
(205 ft) - $185.00 per l.f. of structure

METHOD & SEQUENCE OF CONSTRUCTION

Gabion baskets were assembled at the contractor's yard and brought to the job site. Slope was graded and the filter cloth was laid vertically down the slope with 1' overlap. The gabions were laced together in a 5 basket unit and set in place by the gradall. A 1 ft layer of stone was placed in all 5 baskets, then the next 5 basket unit was placed. All gabion basket lids were closed and the open sides laced down.

GABION REVETMENT



This revetment, approximately 200 feet in length, is constructed of gabions 3 ft. x 12 ft. x 1.5 ft. thick, placed on filter cloth on a 1:1.5 slope. This revetment consists of an upper portion of gabions placed on slope, side by side, with their major axis perpendicular to the beach and a single row of gabions at the toe of the structure running perpendicular to the upper portion. The gabions are filled with stone, 3" to 8" in diameter.

The toe of the structure is placed below grade to minimize damage due to scour. The top elevation of the structure is approximately +10.0 D.R.D. The top portion of the structure is graded on a 1% slope.

NYLON SANDBAG GROINS

MATERIALS

Quantity:	total bags 42 bags/groin 28 bag bottom layer 14 bag top layer
Description:	5' x 10' x 20' (filled) PVC coated sandbags, self closing
Supplier:	Erosion Control Inc. West Palm Beach, Florida
Fill material:	4 tons clean sand/bag

EQUIPMENT

1 front end loader
1 double feed hopper sand pump
1 Bucyrus Erie 30B, 35T Crane w/nylon slings

LABOR

1 Superintendent
1 Operator
2 Dockbuilders
2 Laborers
Total Labor - approx. 240 manhours

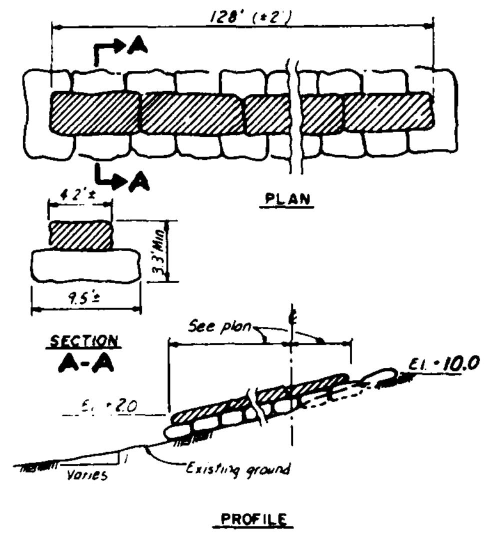
COST

Contract amount = $45,369
360 ft. @ $126/per l.f of structure

METHOD & SEQUENCE OF CONSTRUCTION

Sandbags were pumped full at the work location on wooden pallets with slings in place. The double hopper system enabled twobags to be pumped simultaneously. After filling, the crane set sandbags as close as possible into place.

NYLON SANDBAG GROINS



Each groin is 120 feet long spaced at 180 feet intervals. The groins are placed on the existing grade and have a top elevation at approximately +10.0, Delaware River Datum (D.R.D.), with the bottom approximately at elevations +2.0. The bags, which measure 9.5' long x 4.2' wide x 1.7' thick, are filled with washed sand. Each groin consists of two layers of bags. The bottom row of bags are placed side by side, with their major axis perpendicular to the shoreline. The top row of bags are placed end to end with their major axis perpendicular to the bottom row. In addition, at the landward end, three bags were placed end to end with their major axis perpendicular to the shore. This was done to extend the groin to ground elevation +10.0 D.R.D. to eliminate scour at this end.

GREEN RIVER NEAR
KENT, WASHINGTON

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

GREEN RIVER NEAR KENT, WASHINGTON
DEMONSTRATION PROJECT PERFORMANCE REPORT

INTRODUCTION

1. Project Name and Location. The Green River, Washington, Streambank Protection Demonstration Project is located on the left (west) bank of the Green River near Kent, Washington. A vicinity and location map are shown on sheet 1 of inclosure 1.

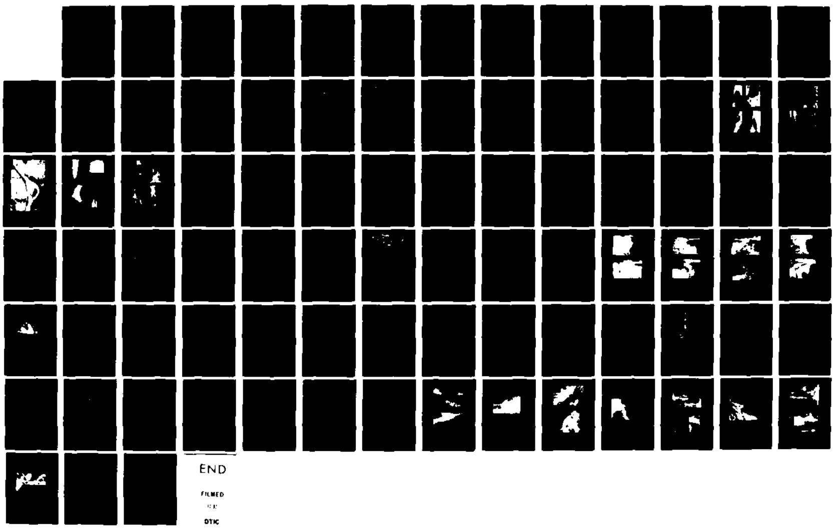
2. Authority. The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251, authorized the Secretary of the Army, acting through the Chief of Engineers, to establish and conduct a national streambank control and demonstration program. The program consists of:

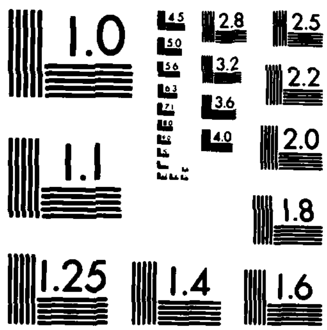
- evaluating the extent of streambank erosion on navigable rivers and their tributaries;
- developing new methods and techniques for bank protection, researching soil stability, and identifying the causes of erosion;
- reporting to Congress the results of such studies and the recommendations of the Secretary of the Army on means for preventing and correcting streambank erosion; and
- constructing demonstration projects, including bank protection works.

3. Purpose and Scope. This report summarizes a bank erosion problem on the Green River near Kent, Washington; describes the bank protection

AD-A121 138 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

methods used to control the erosion; reviews the monitoring of the project following construction; and presents evaluations and conclusions of the erosion control measures. The report covers preconstruction conditions, construction activities, and postconstruction maintenance and monitoring. Involvement and coordination with local, county, state, and other Federal agencies are highlighted.

4. Problem Resume. About 1,000 lineal feet of the left (west) bank of the Green River were nearly vertical and actively eroding at a rate of several feet per year. The riverbank upstream and downstream of the 1,000-foot reach was also eroding but at a lesser rate. Local King County government was interested in stopping the erosion and supported the demonstration project as a means to provide needed bank protection. King County owns a 150-foot-wide strip of property adjacent to the riverbank along the project and all of the property in the river bend north of the project. The King County Parks Department plans to develop their property into a recreation park in the future. Property west and south of the project is expected to be developed into high density residential in the future. Photographs 1, 2, 3, and 4 show the preconstruction bank conditions.

HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The lower Green River Valley consists of a 2- to 3-mile-wide flood plain bounded on the east and west by relatively steep valley walls rising several hundred feet. The river meanders within this flood plain, and it is within one of these meanders, near the east valley wall, that the site is located. Property landward of the riverbank, along the project reach, is level with only a few feet of surface relief. Before construction, existing streambank slopes ranged from vertical to 1 vertical on 1 horizontal due to ongoing sloughing and erosion.

b. Geology. The valley is the remnant of a glacial trough occupied by stagnating ice during the closing stages of continental glaciation about 12,000 years ago. The glacial deposits under the valley floor are now well below sea level and are overlain by a series of delta-marine sands and gravels essentially deposited by the White River aggrading the trough. These sediments are graded finer upward in the section and downstream from where the White River enters the trough at Auburn. The present surface of the valley is alluvium, crudely graded to that point. Glacial deposits also comprise the material in the valley walls, though exposures are often concealed by landslides and other recent colluvium.

c. Hydrologic Characteristics. Green River is controlled by Howard A. Hanson Dam, a Corps of Engineers flood control project located about 64 miles upstream from the river mouth. Howard A. Hanson Dam is operated on a rule curve which in normal years requires the reservoir to be lowered by 1 November to provide flood control during the winter months. Storing begins in May to raise the reservoir to conservation pool elevation by mid-June. Normally, a minimum release of 220 cubic feet per second (c.f.s.) is made during the summer months to meet fisheries and municipal and industrial water supply requirements. Project releases during the flood control season regulate maximum flow to 12,000 c.f.s. (channel capacity through the flood plain) in the vicinity of the project. A typical regulated discharge hydrograph during the flood control season is shown on plate 1 of inclosure 2. The flow-duration curve (under regulated conditions) is shown on plate 2 of inclosure 2.

d. Channel Conditions. From the mouth at Seattle to Auburn, Washington (river mile 32), regulated Green River flows are confined to the channel by high streambanks and riverside levees. The banks are generally covered with native grasses and are relatively stable. Channel cross sections surveyed in 1961 and 1980 show that aggradation/degradation is relatively insignificant in this reach.

e. Environmental Considerations. The project area is classified by the Soil Conservation Service (SCS) as prime farmland. Upland vegetation adjacent to the project consists of pasture, forbs, and grasses. Upstream and downstream of the pasture are woody thickets and overstory typical of riverbanks in this area. Wildlife usage of the project site is limited primarily to a variety of bird species. Steelhead trout and chinook, coho, and chum salmon utilize the Green River system. Due to erosion and the deposition of sandy streambank material, no spawning habitat is thought to exist adjacent to the immediate project area. No threatened or endangered species exist in the project area and no evidence of cultural resources was discovered. The major long-term effects expected on the shoreline habitat as a result of the erosion control measures include more stable bottom conditions and a more gradual shoreline slope along the project rock toe area, increased accessibility to the river edge due to flatter riverbank slopes, greatly reduced erosion and bank sloughing along the project reach, increased cover of shrubs along the project reach, and increased diversity of shoreline vegetation. No significant long-term impacts to water quality are expected. Overall, the project should benefit the habitat of the site by the provision of erosion protection. A finding of no significant impact (FONSI) and an environmental assessment of the project were circulated for public review in October 1979. Following a 30-day review period, the decision not to prepare an environmental impact statement was finalized, and the FONSI was signed and placed in the project files. The National Marine Fisheries Service and Washington State Department of Game commended the design and efforts to develop alternative bank protection methods which are more environmentally acceptable than traditional riprap protection.

6. Demonstration Site - Test Reach.

a. Hydrologic Characteristics. The test site is located about 37 miles downstream from Howard A. Hanson Dam and about 5 miles downstream from the U.S. Geological Survey streamflow gaging station near

Auburn, Washington. No major streams enter the Green River between the gage and the test reach. Additional information is included in paragraph 5 above.

b. Hydraulic Characteristics. Plate 3 of inclosure 2 shows the stage-discharge relationship at the site. Average channel velocities as determined from slope-area computations range from about 2 to 5 feet per second (f.p.s.) for discharges of 2,000 to 12,000 c.f.s., respectively. Local velocities are estimated to range up to 7 f.p.s. Velocity distribution within the cross section is not known. Following high water in February 1981, stream velocities along the project bank were taken and ranged from 2 to -2 f.p.s. with a discharge of 6,600 c.f.s. at the Auburn gage.

c. Riverbank Description.

(1) Bank Materials. Foundation and bank materials at the site consist of uniform, unconsolidated fine sands and silt of alluvial origin. The soils are stratified to some degree, resulting in layers of variable permeability. This fine-grained alluvium probably extends to depths on the order of 50 feet in the vicinity of the project.

(2) Normal Bank Vegetation. Preconstruction vegetation cover along the project reach varied from top to bottom and from end to end but consisted largely of grasses. The flatter upper portions near the top of the bank reflected vegetation typical of the heavily grazed pasture adjacent to the project reach and was composed primarily of disturbed site grasses dominated by orchard grass, soft brome-grass, perennial ryegrass, and large clumps of Columbia brome grass interspersed with clover and edible thistle. The drier, steeper portions of the project reach contained grasses dominated by spike and creeping bentgrass, orchard grass, perennial ryegrass, and velvet-grass interspersed with clumps of thistle and hawkweed. Vegetation of the moist, steep portions of the slope below the midpoint of the bank was dominated by horsetail

interspersed with thistle and some orchard grass, soft brome-grass, and chickweed. Riparian vegetation (2 feet above the waterline down to mudflats) was characterized by clumps of velvet-grass, perennial ryegrass, and creeping bentgrass within large patches of reed canarygrass. A few small willows and alders grew along the base of the project reach. Vegetation along the banks of the Green River upstream and downstream from the project reach consist of woody thickets and overstory, including willow, cottonwood, and maple.

(3) Bank Erosion Tendencies. The sand and silt bank materials in the project area are highly susceptible to erosion from ground water, surface runoff, and streamflows. Generally, river bends along the Green River that do not have well-established vegetation or rock bank protection are either actively eroding or have in the past been seriously eroded. The project site is located in a bend of the river where the native vegetation protection has been destroyed and active erosion has prevented reestablishment of natural protection. The combination of ground water seepage exiting on the bank and high river stage usually caused the greatest erosion losses of the bank.

DESIGN AND CONSTRUCTION

7. General. Three separate erosion control measures, outlined in detail in paragraph 9, have been used. Each measure is used twice, once on a section of riverbank where erosion potential was expected to be low and again in a section where erosion potential was expected to be high. Each section is separated by a 4-foot-wide by 4-foot-thick section of riprap. A 100-foot-long section of 18-inch-thick riprap is constructed at each end of the project to transition from the steep natural embankment to the graded slopes of the project. An 18-inch-thick riprap blanket with a weighted toe is placed on the slope below ordinary high water (OHW) along the entire project length. Shrubs and native grasses are planted on all test sections. Plan and typical sections are

outlined on sheets 2, 3, 4, and 5 of inclosure 1. Photographs 5, 6, 7, and 8 show the project following construction.

8. Basis of Design.

a. OHW. The OHW line is used as the lower boundary of the vegetation plantings on each test section. The OHW line is established at the approximate base elevation of woody plants on the opposite bank. The Green River water surface equals or exceeds OHW about 20 percent of the time in an average year.

b. Riprap and Filter. That portion of the streambank located below OHW is protected with an 18-inch-thick blanket of riprap and a 2-foot-thick by 6-foot-long weighted toe to prevent streamflow from eroding the toe of the bank and exposing the test portions of bank from failure by undermining. The riprap is designed by the tractive force methods outlined in EM 1110-2-1601 and ETL 1110-2-60. The gradation used is based on a safety factor of 1.75. A 1-foot-thick quarry spall blanket is placed as bedding under the riprap blanket.

c. Embankment Slope. The embankment is sloped to 1 vertical on 2 horizontal below OHW for stability. Above OHW, the embankment slope is 1 vertical on 3 horizontal to permit placement and maintenance of shrubs and grass. Waste material from excavation was disposed of in existing depressions in the proposed county park site near the project.

d. Plantings. A project objective is to evaluate the effectiveness of vegetation to control erosion on embankment or levee slopes in an effort to develop more environmentally acceptable erosion control measures. The project reach is planted with four species of native shrubs to be monitored for their response to different soil treatments and erosion forces. Planting configurations were chosen for a "controlled" research project. All project sections are seeded with native grasses.

9. Construction Details.

a. Planting Plan. Shrubs have been planted 2 feet on center in all six test sections. The planting plan used is shown on sheet 5 of inclosure 1. Each test section has five test plots of about equal length. Four of the plots have been planted with the four shrub species, one species per plot. The fifth plot is a mixture of the four species. Location of the five plot types is random within each test section and each differs somewhat. In the mixed plots, the plants have been placed in columns of all one species, but the order of the species is random. The shrubs were propagated and planted by the SCS after earthwork and riprapping was completed. Following planting of the shrubs, the entire bank slope was hydroseeded with 40 pounds/acre native grass seed mixed with 500 pounds/acre wood fiber mulch.

b. Plant Species. Four native shrub species that are adapted to river edge environments have been used: dogwood (*Cornus Stolonifera*), snowberry (*Symphoricarpus albus*), spirea (*Spirea douglasii*), and willow (*Salix lasiandra*).

c. Erosion Control Measures.

(1) Wire Mesh. Steel fabric wire netting of about 1-inch mesh has been placed on the graded embankment in sections 2 and 4, as shown on inclosure 1, to reduce initial surface erosion while plantings become established. Shrubs were planted through the wire netting.

(2) Quarry Spalls. A 2-foot-thick blanket of mixed quarry spalls and native backfill was placed on the graded embankment in sections 3 and 6. Due to voids left following placement, additional topsoil was added to cover the quarry spalls. In theory, quarry spalls give initial added protection and ultimately the spalls, soil, and plantings should form an interlocked protective blanket.

(3) Native Embankment. Erosion control in sections 1 and 5 results from resloping the bank and adding plantings. This measure is the most "natural."

d. Construction Methods. The bank slope was constructed by excavating the bank material by dragline and transporting the waste for disposal by dump truck. Riprap, quarry spalls, and the quarry spall-native material mixture were placed by dragline using a skip. The quarry spall-native material mixture was premixed prior to placement. Additional topsoil was placed over the placed mixture to fill the voids. The final grading of the bank slope was accomplished using a small crawler tractor. Earthwork construction began on 1 August 1980 and was completed on 6 October 1980. Shrubs (17,000) were planted between 7 October and 20 October 1980 by a six-man SCS crew. On 20 October 1980 the project was hydroseeded.

10. Cost. The project cost is summarized below:

a. Initial Planning Design and Construction.

(1) E&D (including overhead).

Basis of design report (includes Coordination, EA, 404 Evaluation, Shoreline Management Coordination, Local Sponsor Coordination)	$37,000
Preparation of Plans and Specifications (includes earthwork plans, coordination of SCS shrub propagation agreement and hydroseeding contract)	$80,000
Engineering After Award	$3,000
Total E&D (initial construction)	$120,000

(2) Construction.

Earthwork Contract (includes S&I)	$245,000
Propagation and Planting of Shrubs	15,000
Hydroseeding Contract	2,000
Total Initial Construction	$262,000

b. Post Construction.

(1) E&D

Monitoring	$20,000
Coordination and Design For Damage Repair	10,000
Total	$30,000

(2) Emergency Repairs

Project Access Road	$30,000
Earthwork Repair	34,000
Propagation and Planting of Shrubs	4,000
Hydroseeding	2,000
Total	$70,000

c. Additional funds have been requested to complete project through FY 81 (monitoring and maintenance of repair work) and turn over to King County.	$20,000
d. Total Project Cost	$502,000

A breakdown of estimated initial construction costs on a per foot basis for the three types of sections is shown below.

Native plantings on graded slope	$81/foot
Steel fabric and native plantings on graded slope	$97/foot
Quarry spalls mixed with native fill and native plantings on graded slope	$145/foot

PERFORMANCE OF PROTECTION

11. Monitoring Program.

a. Monitoring Plan. Postconstruction monitoring of the project began on 20 October 1980 and officially terminated for this report on 20 March 1981. Monitoring has consisted of visual observations; photographic records; survey cross sections; hydraulic evaluation of velocity, discharge, and aggradation; soils and plant material evaluations; and botanical evaluations. The monitoring program was developed for a 3- to 5-year period but will be formally terminated at the end of Fiscal Year 1981 when the statutory authority for Section 32 expires. The monitoring program developed for the project is described in inclosure 4. Although some usable data has been obtained in the 5 months of monitoring, insufficient time has elapsed on which to fully evaluate the viability of the protection methods.

b. Events Monitored. Several significant events have occurred during the 5 months of monitoring and are discussed below. Photograph 9 shows the project in mid March 1981.

(1) November 1980 Rainstorm. Between 3 and 8 November 1980, heavy rains occurred in the project area and caused severe gullying of the slope from a combination of ground waterflows piping the more permeable sand layers and surface runoff. Much of the runoff was generated

from the area at the top of the slope where native grasses had been removed during construction activities resulting in rapid concentrations of surface water. Photographs 10, 11, 12, and 13 show typical erosion damage following the rains. The surface runoff problem was corrected in late November by construction of a berm at the top of the slope and regrading the area above the slope to drain away from the project or to pipes placed down the slope. Eroded areas of the bank slope were filled with gravel fill.

(2) December 1980 High Water. In late December 1980, discharges up to 7,500 c.f.s. occurred at the project and raised the water surface about halfway up the test slope. When the water receded, erosion had occurred along the downstream section with steel fabric native plantings, and the downstream section with quarry spalls and native plantings. In the quarry spall section, the topsoil, shrubs, and grass were eroded from the quarry spalls along most of the length of the section and about halfway up the slope between the riprap and the top of the bank. In the steel fabric section, the bank was eroded above the riprap along about 75 percent of the length of the section and about 6 feet in elevation above the riprap. The discharge hydrograph is shown on plate 4 of inclosure 2.

(3) February 1981 High Water. In February 1981, discharges up to 9,300 c.f.s. occurred at the project site and caused additional erosion to the downstream section with steel fabric. A large tree lodged on the riprap along the section and aggravated the erosion. About 30 percent of the section was eroded back to the top of the bank slope with a nearly vertical face. Shrubs and grass on the eroded areas have been lost. Photographs 14, 15, 16, and 17 show the erosion damage. The discharge hydrograph is shown on plate 5 of inclosure 2.

12. Evaluation of Protection Performance. Construction of the project was originally scheduled for June 1980 but was delayed to August 1980 by funding constraints. By constructing the project in late summer instead

of early summer, grass cover had little chance of becoming established and the light grass cover that developed between October and November 1980 did not provide effective erosion protection. In addition, the wire mesh did not provide expected interim erosion protection until grass could become established. Because a grass cover was not established and shrubs were not mature, no evaluation of the protection provided by grass and shrubs can be made at this time. To date, the quarry spall-native material mixture prevented severe slope damage, but substantial erosion of the native material from the mixture has occurred, leaving large voids in the blanket. Shrubs have shown good survival except where lost from erosion. Continued informal monitoring should establish the effectiveness of the grass after a full summer's growth and effectiveness of shrubs as they mature. Riprap constructed at each end of the project and below OHW along the toe of the test sections provided good erosion protection and was not damaged during the monitoring period.

13. Rehabilitation. Damages were repaired in May and early June 1981. Since the Section 32 program is ending, the reconstruction was designed to be permanent and structurally sound before the local sponsor, King County, assumed maintenance responsibility. Replacement of the downstream section with wire fabric as originally designed, even with a full summer for grass to become established, was not expected to provide adequate erosion protection based on the limited experience gained thus far. The repair consisted of filling the eroded area with native fill, construction of a toe drain and protecting slope with quarry spalls. Gravel bedding was placed under the spalls and topsoil was placed over the spalls. All other eroded areas were repaired with gravel fill, covered with topsoil, grass seeded, and lost shrubs were replaced. The wire mesh, mostly destroyed in February 1981, was removed from the damaged portion of the section. Reconstruction plans were coordinated with King County and accepted in April 1981.

14. Conclusion. Effectiveness of the methods of protection tested cannot be fully evaluated due to the short period of time (5 months) since monitoring began. The vegetative cover did not have sufficient time to become established prior to being subjected to the flood season, so no conclusion can be made regarding its effectiveness. Experience gathered from the short period of monitoring suggests the following items should receive specific consideration in design of bank protection utilizing vegetative cover:

- Construction should be scheduled so that plants and grass have maximum possible time prior to the flood season to become established. On the Green River, construction should be scheduled in early spring because the flood season begins in the fall.

- Dependable year-round access was added to the Green River project when damage required access by construction equipment during wet weather and the "fair weather" road became impassable. The basic design did not anticipate access problems but should more closely evaluate access during the flood season.

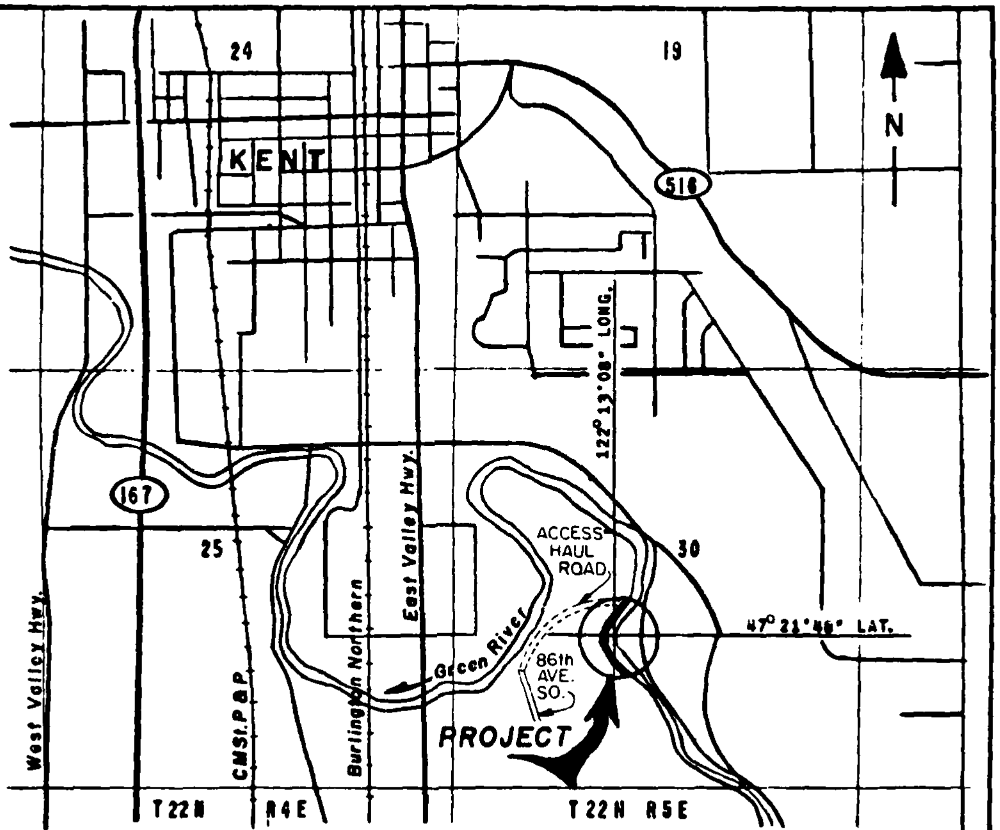
- Surface runoff from the adjacent flood plain should be diverted away from the bank or structurally conveyed down the test slope to prevent slope erosion until the vegetation becomes established.

- Migration of ground water from the adjacent flood plain to the river channel, especially when the local water table is perched because of impermeable soil strata, should be considered and filter materials used to prevent piping of material from the streambank. Design should consider both seepage and stream flow when evaluating the need for filter material under quarry spalls and rock.

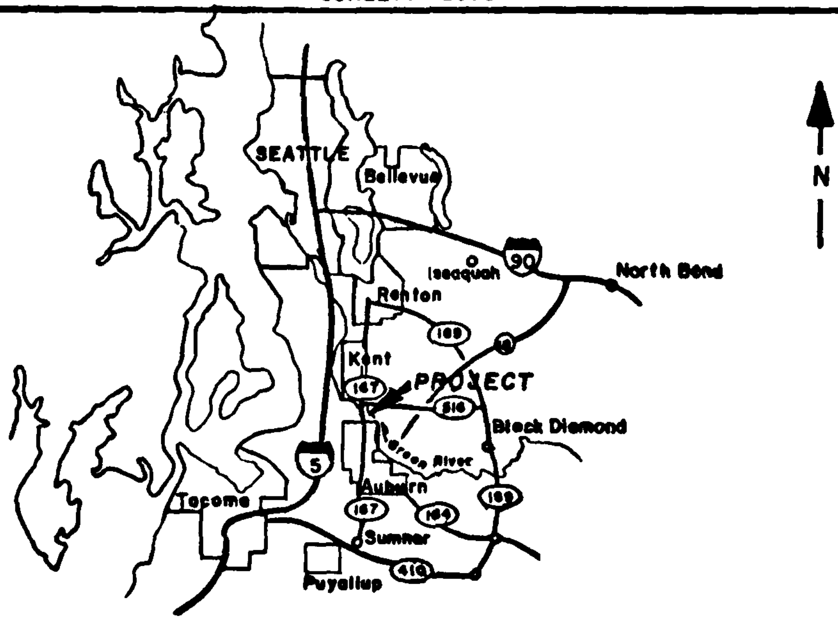
- The native soil-quarry spall mixture and plantings used did not provide material capable of resisting streamflow erosion. Premixing quarry spalls with topsoil did not provide a homogenous mixture and

resulted in voids and unstable spalls when the native soil was eroded. Whether the material would have preformed adequately if the grass and plantings had 6 to 8 months to become established is conjectural.

- The presence and effect of stream carried debris on slope protection must be considered in design.



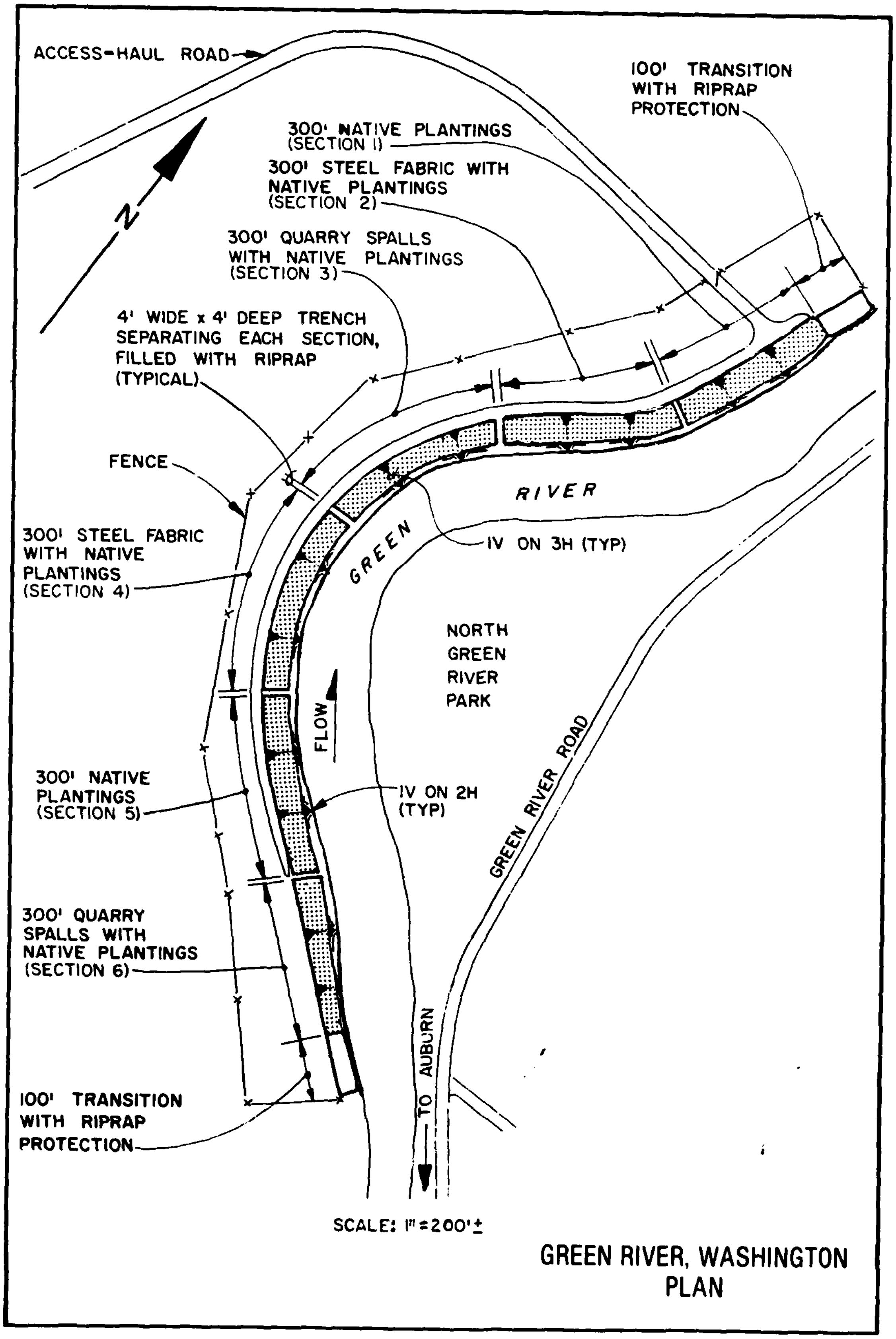
LOCATION MAP
SCALE: 1" = 2000'



VICINITY MAP
SCALE: 1" = 17.5 MILES

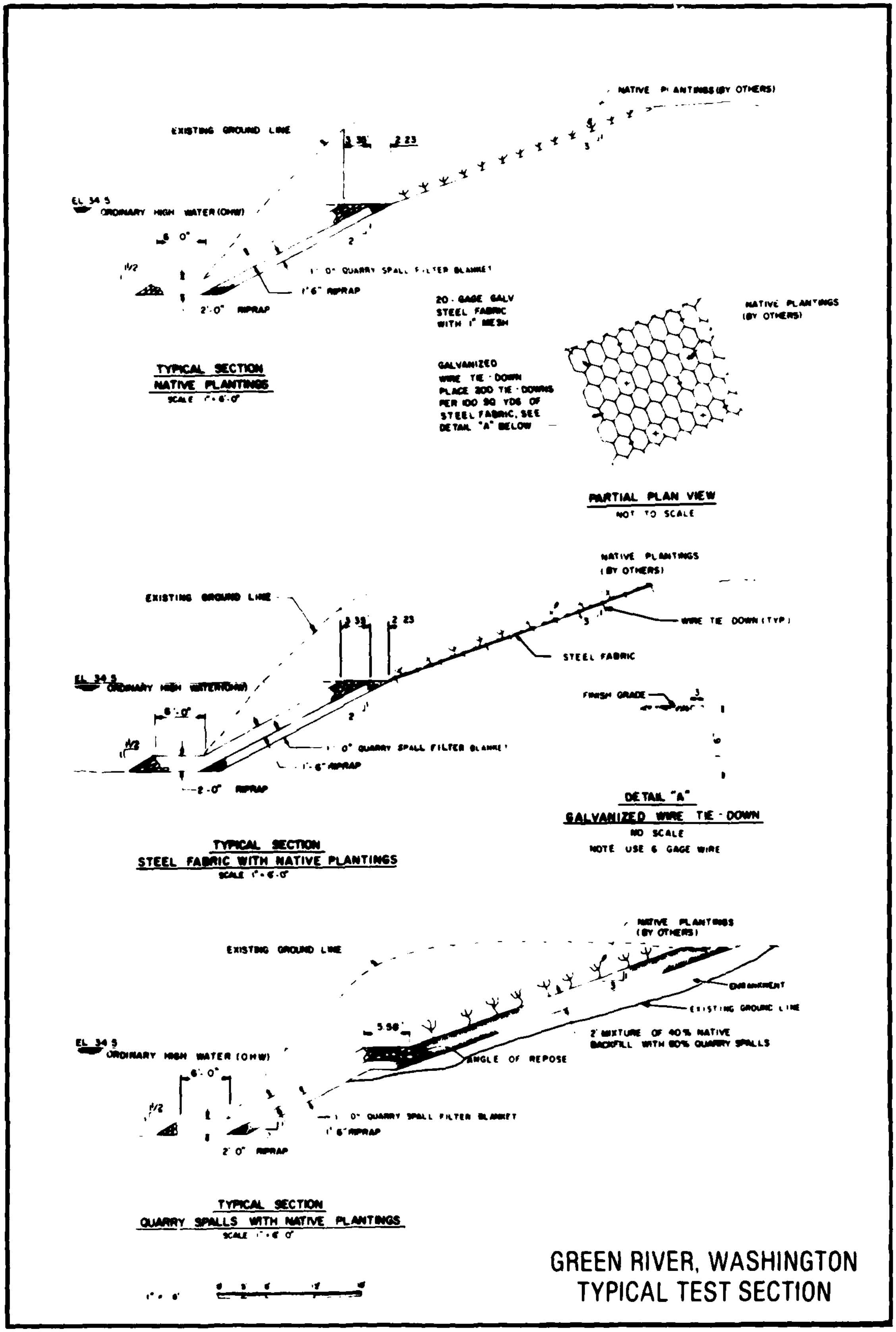
GREEN RIVER, WASHINGTON
LOCATION AND VICINITY MAPS

INCLOSURE 1 (SHEET 1 OF 5)

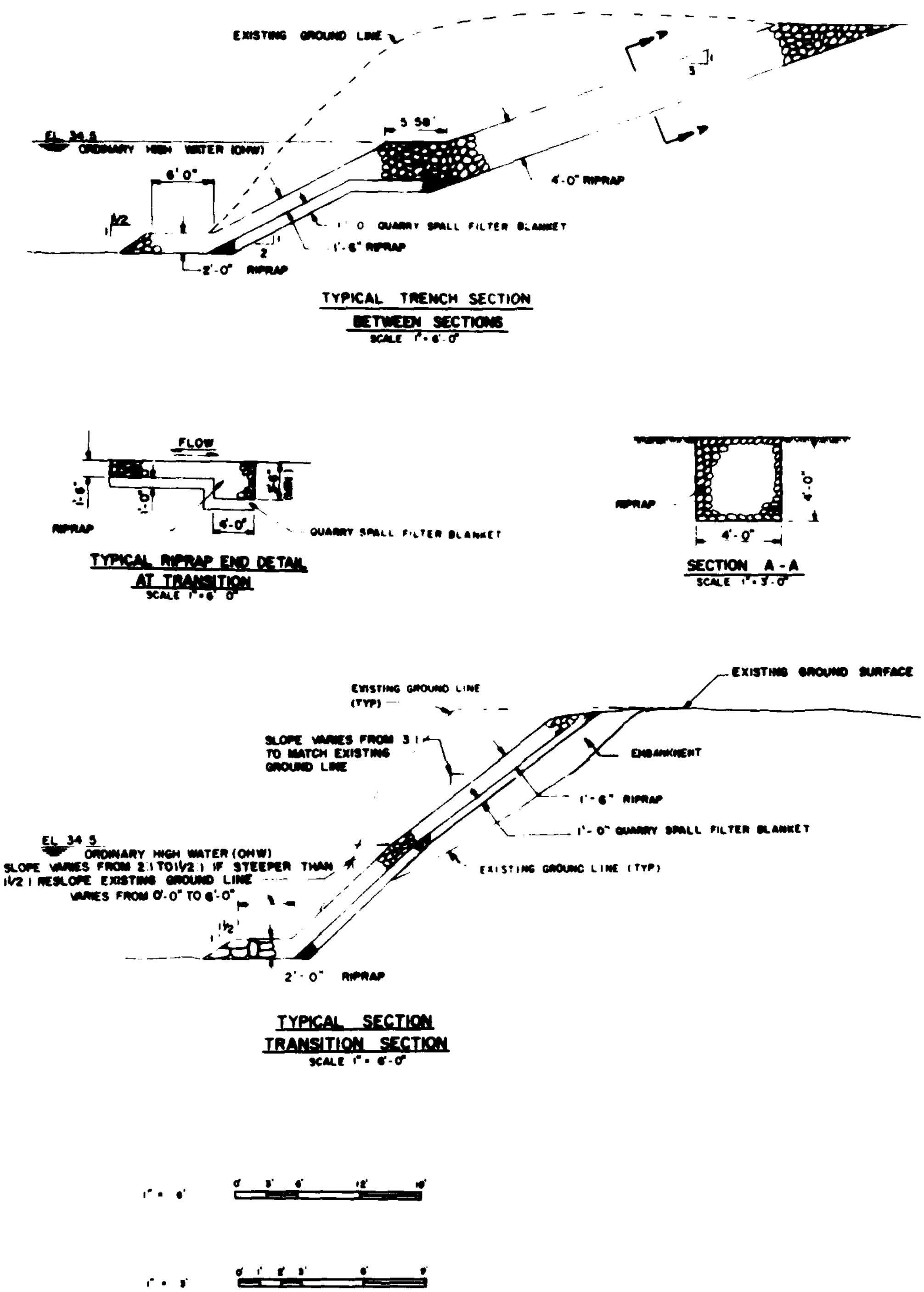


GREEN RIVER, WASHINGTON
PLAN

INCLOSURE 1 (SHEET 2 OF 5)

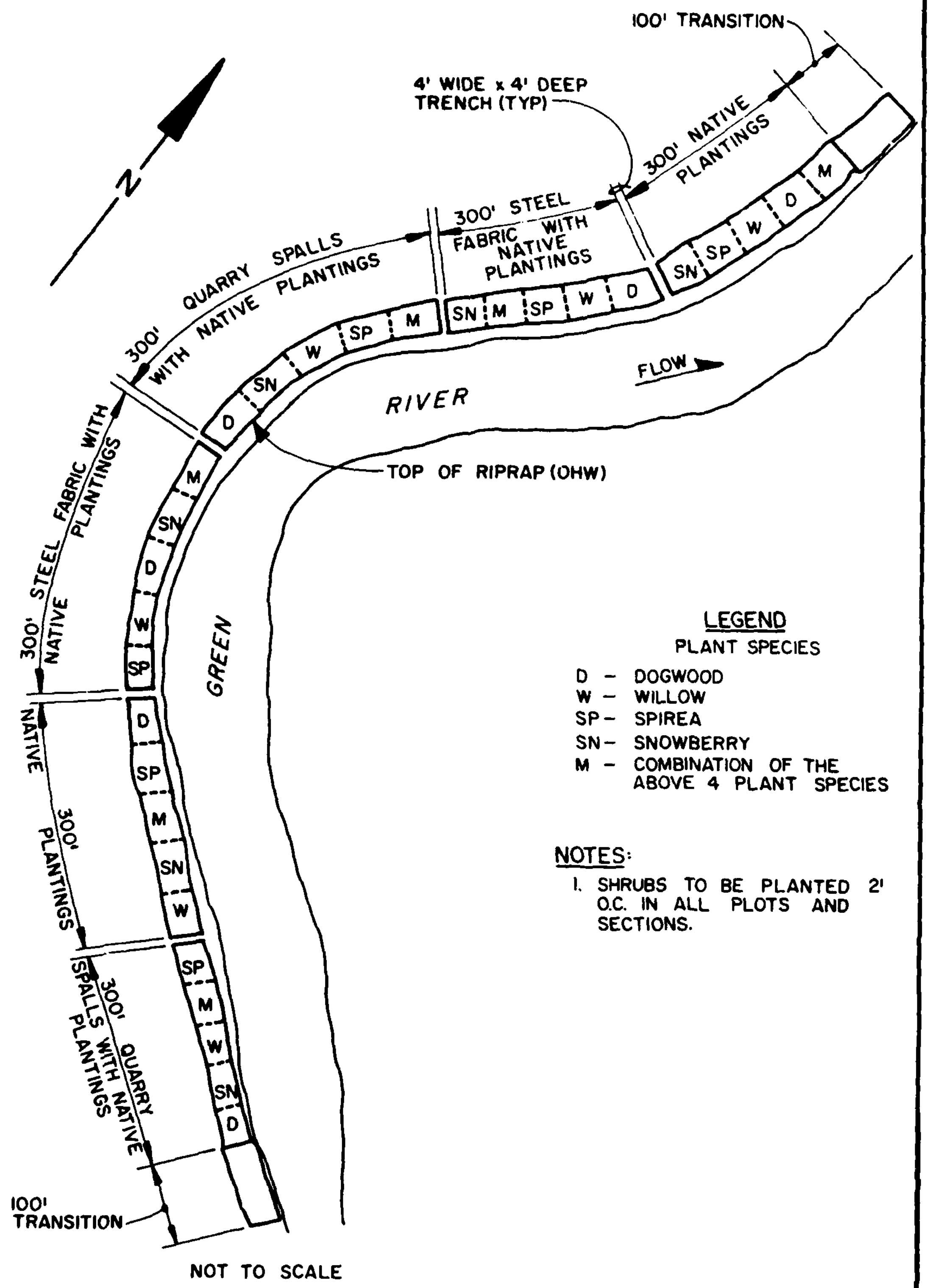


INCLOSURE 1 (SHEET 3 OF 5)



GREEN RIVER, WASHINGTON
SECTIONS AND DETAILS

INCLOSURE 1 (SHEET 4 OF 5)



LEGEND

PLANT SPECIES

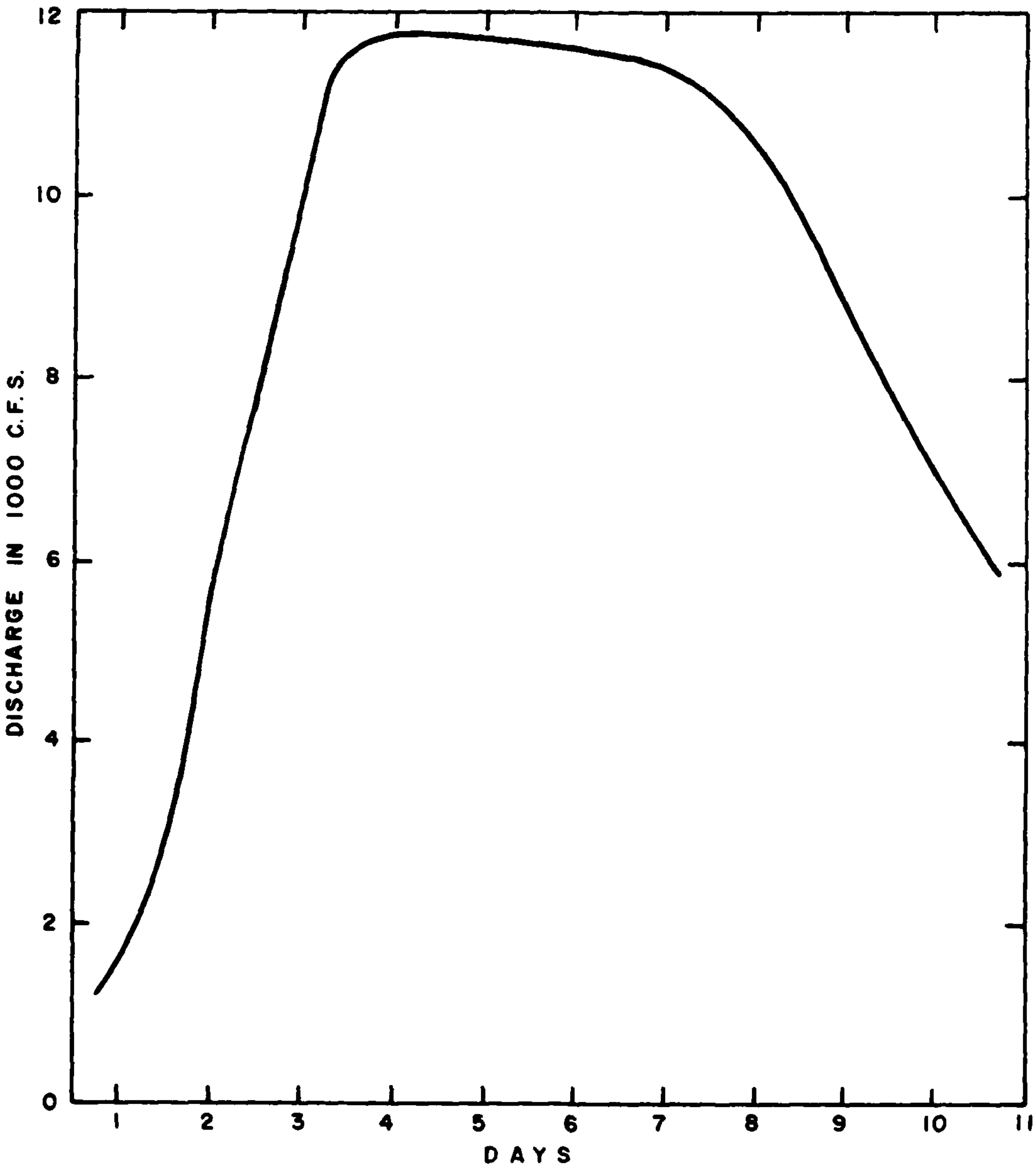
D – DOGWOOD
W – WILLOW
SP – SPIREA
SN – SNOWBERRY
M – COMBINATION OF THE ABOVE 4 PLANT SPECIES

NOTES:

1. SHRUBS TO BE PLANTED 2' O.C. IN ALL PLOTS AND SECTIONS.

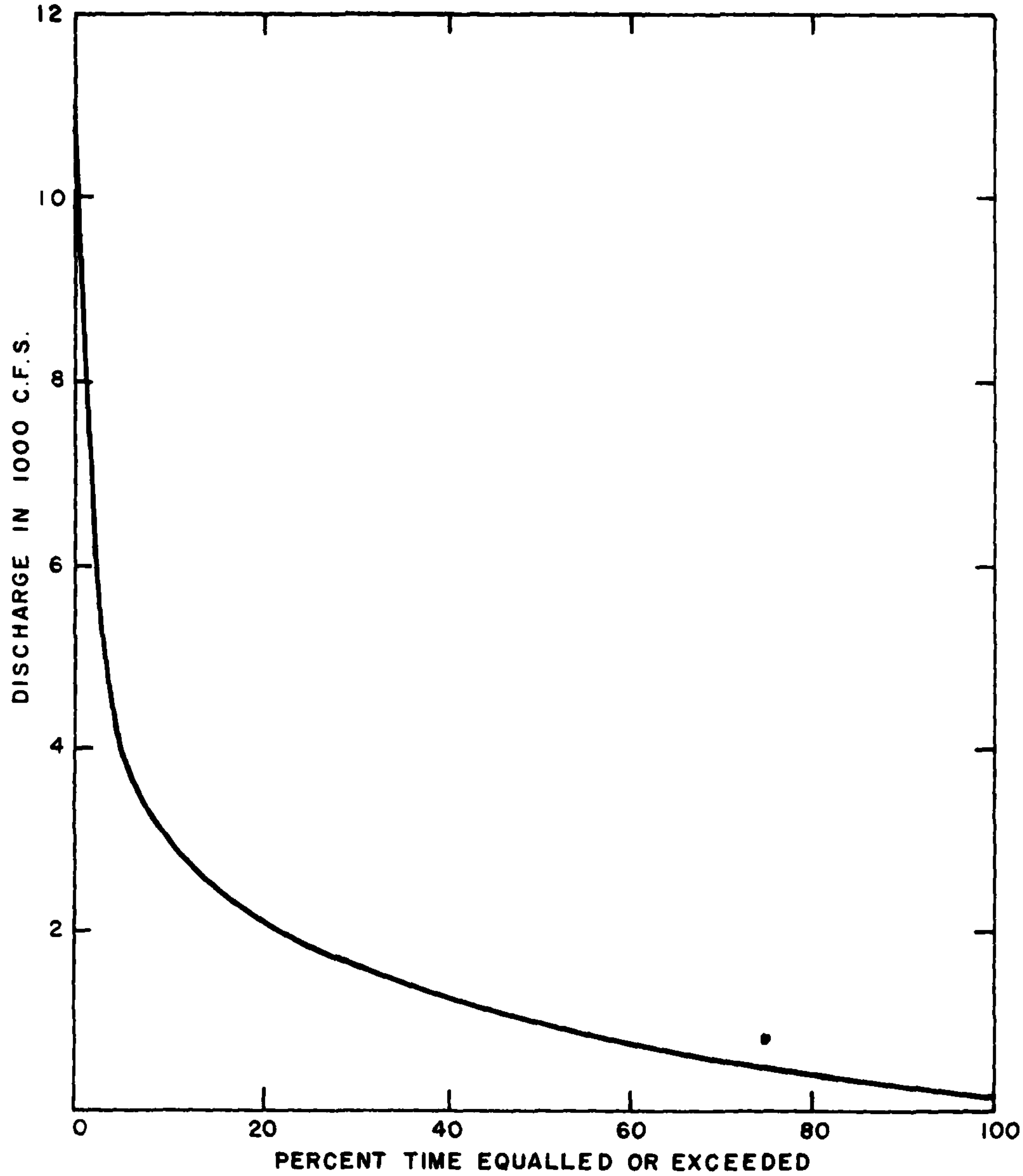
GREEN RIVER, WASHINGTON PLANTING PLAN AND DETAILS

INCLOSURE 1 (SHEET 5 OF 5)



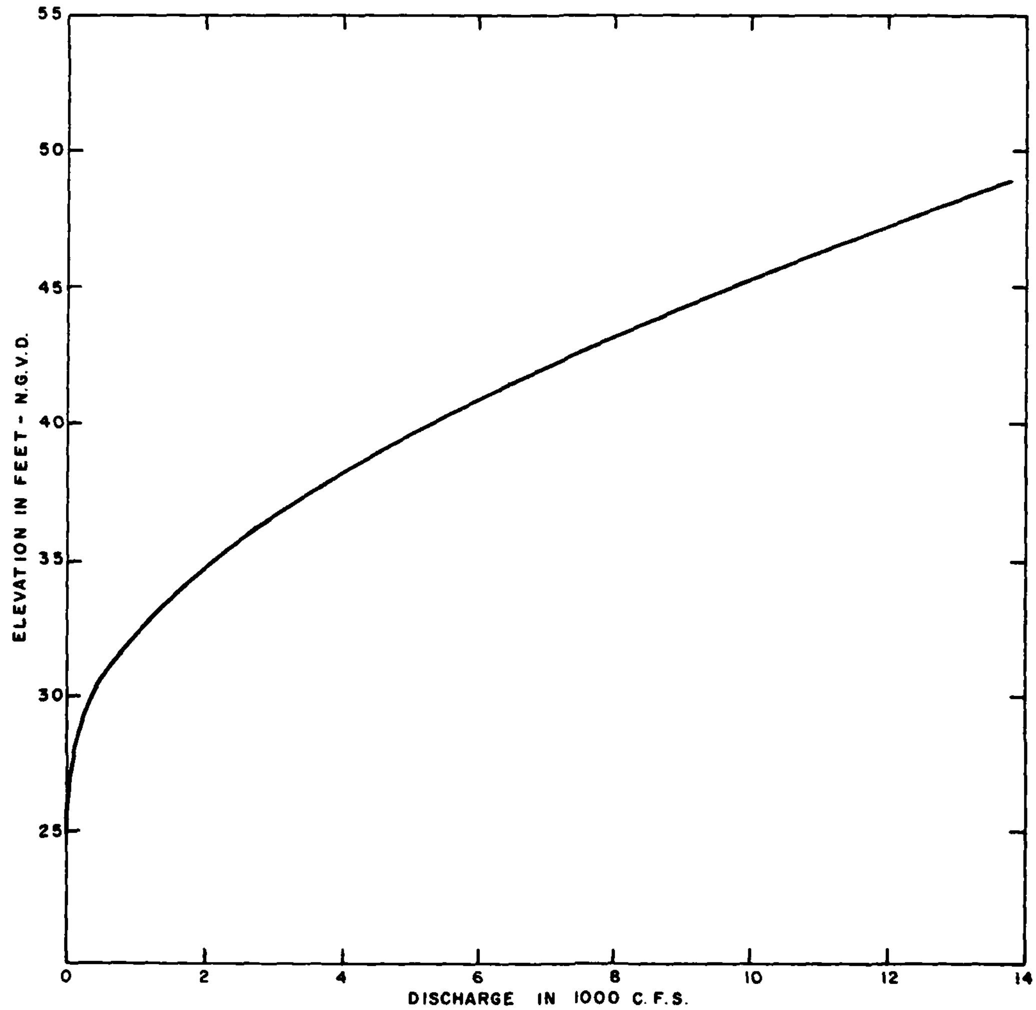
HOWARD HANSON DAM
TYPICAL REGULATED
DISCHARGE HYDROGRAPH

INCLOSURE 2, PLATE 1



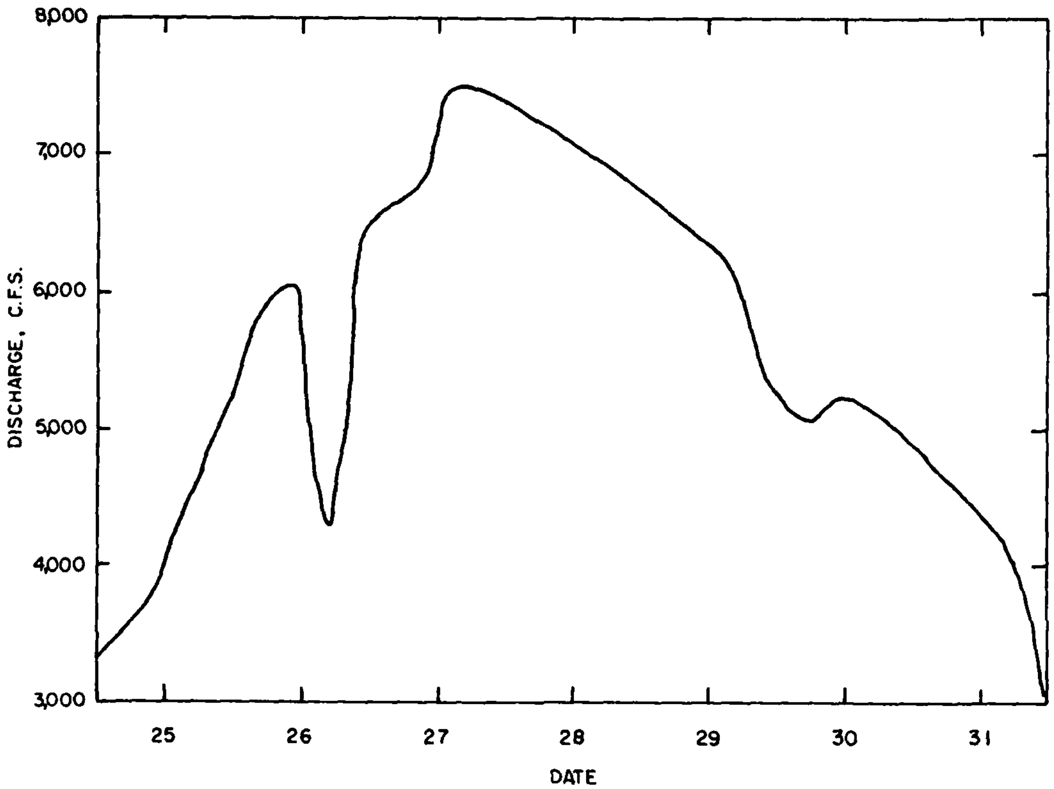
GREEN RIVER, WASHINGTON
STREAM GAGE NO. 12-1130 NEAR AUBURN
FLOW DURATION CURVE
(REGULATED CONDITION)

INCLOSURE 2, PLATE 2



GREEN RIVER, WASHINGTON
NEAR STREAMBANK DEMONSTRATION PROJECT
ELEVATION VS DISCHARGE CURVE

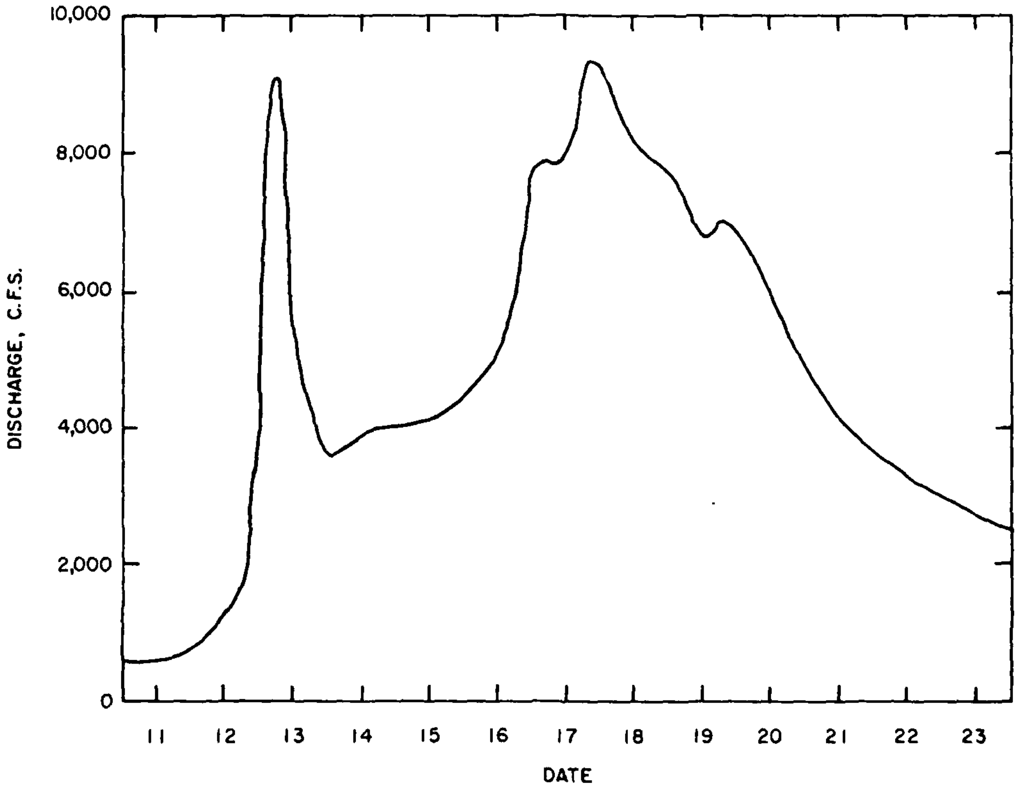
INCLOSURE 2, PLATE 3



DISCHARGE HYDROGRAPH* AT AUBURN GAGE
DECEMBER 1980 FLOOD

* BASED ON PROVISIONAL USGS GAGE DATA.

INCLOSURE 2, PLATE 4



DISCHARGE HYDROGRAPH* AT AUBURN GAGE
FEBRUARY 1981 FLOOD

* BASED ON PROVISIONAL USGS GAGE DATA.

INCLOSURE 2, PLATE 5

PHOTO 1. AERIAL VIEW LOOKING WEST. PRE-CONSTRUCTION, OCTOBER 1979.

PHOTO 2. PRECONSTRUCTION LOOKING UPSTREAM, JULY 1980.

PHOTO 3. PRECONSTRUCTION LOOKING UPSTREAM, JULY 1980.

PHOTO 4. PRECONSTRUCTION, TYPICAL ACTIVE EROSION, JULY 1980.

GREEN RIVER, WASHINGTON
PRECONSTRUCTION CONDITIONS

INCLOSURE 3, PHOTOS 1-4

PHOTO 5. LOOKING DOWNSTREAM AT SECTIONS 3 AND 2, OCTOBER 1980.

PHOTO 6. LOOKING UPSTREAM AT SECTIONS 2, 3, AND 4, OCTOBER 1980.

PHOTO 7. LOOKING UPSTREAM AT PLANTINGS ON SECTION 2, OCTOBER 1980.

PHOTO 8. LOOKING UPSTREAM AT TRANSITION ON SECTION 1, OCTOBER 1980.

GREEN RIVER, WASHINGTON
POST CONSTRUCTION

INCLOSURE 3, PHOTOS 5-8

PHOTO 9. AERIAL VIEW OF PROJECT IN MARCH 1981.

GREEN RIVER, WASHINGTON

INCLOSURE 3, PHOTO 9

PHOTO 10. SURFACE RUNOFF, ERODED A GULLY IN SECTION 1, NOVEMBER 1980. EROSION IS 3' DEEP × 8' WIDE × 15' LONG.

PHOTO 11. TYPICAL EROSION FROM SURFACE RUNOFF, NOVEMBER 1980.

PHOTO 12. TYPICAL EROSION FROM SURFACE RUNOFF, SECTION 2, NOVEMBER 1980.

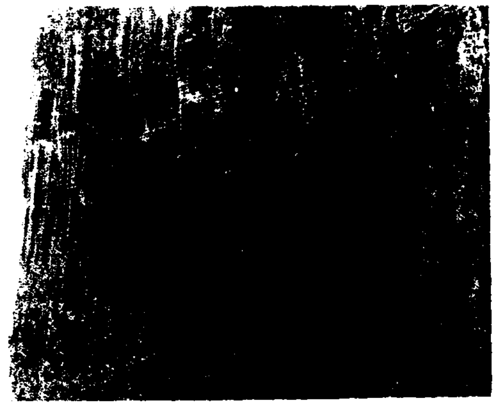


PHOTO 13. TYPICAL OF TWO EROSION AREAS IN SECTION 4, NOVEMBER 1980. EROSION IS UP TO 4' DEEP × 5' WIDE × 20' LONG.

GREEN RIVER, WASHINGTON
DAMAGE FOLLOWING NOVEMBER 1980 RAIN STORM

INCLOSURE 3, PHOTOS 10-13

PHOTO 14 LOOKING UPSTREAM ALONG SECTION 2
PHOTO SHOWS EROSION AND DEBRIS
FROM FEBRUARY 1981 HIGH-WATER.
MARCH 1981

PHOTO 15 LOOKING UPSTREAM ALONG SECTION 3.
SECTIONS 4 AND 5 ARE IN BACKGROUND.
MARCH 1981.

PHOTO 16 LOOKING DOWNSTREAM AT EROSION AT
QUARRY SPALLS - NATIVE MATERIAL
BLANKET IN SECTION 4 AND EROSION IN
SECTION 2. MARCH 1981.

PHOTO 17 LOOKING UPSTREAM AT EROSION IN
SECTION 2. MARCH 1981

GREEN RIVER, WASHINGTON
FOLLOWING FEBRUARY 1981 HIGH WATER

INCLOSURE 3, PHOTOS 14-17

GREEN RIVER STREAMBANK PROTECTION DEMONSTRATION PROJECT
KENT, WASHINGTON
PROJECT MONITORING PROGRAM

1. Introduction. The project monitoring program will conform to "Guidelines for Monitoring and Reporting Demonstration Projects - Section 32 Program," instruction report H-77-1. The program is scheduled for a 2-5 year period. General details of the monitoring program are described in subsequent paragraphs. The general purpose of the program is to collect data which will serve as a basis for evaluating the functional behavior of the protection methods tested and the conditions to which it has been exposed.

2. Data Collection. The following items of data collection will be included in the monitoring program at the specified time intervals.

WORK ITEM	FREQUENCY
a. Field Data of Physical Features (Survey Branch)	
(1) Full channel cross sections every +110 feet through length of project (16 sections). One section upstream of project and one section downstream of project.	Annually following construction.
(2) Permanently monument each cross section so following years sections can be taken at the same locations.	Following construction.
(3) Aerial photographs of project site in color and low elevation, including vertical and obliques.	Preconstruction and annually following construction.
(4) Detailed topographic mapping, 2-foot contour interval, for length of project and 100 feet landward from each riverbank	Preconstruction.
b. Hydrologic Data (Hydraulics and Hydrology Branch)	
(1) Velocity distributions for three different discharges (low, average, high). Each flow consists of velocities at four locations across the river of at least three depths. These separate sections along the project will be tested for each discharge.	Following construction and as required in following years.
(2) Discharge rating curve	Initially-Annual point check. Update curve as required.

INCLOSURE 4 (SHEET 1 OF 5)

WORK ITEM	FREQUENCY
(3) Record of abnormal water level and duration	As condition occurs.
(4) Local climatic conditions, local agradation conditions, river traffic, wave action	Annual summary.
c. Material (Foundation and Materials Branch)	
(1) Soils samples, classifications for soil profile at each test section	Preconstruction.
(2) Visual observation of soil material performance and update materials analysis. Indicate stability of bank, structural integrity and durability of material, erosion conditions	Annually, following highwater.
d. Plant Material (Civil Design Landscaping Unit)	
(1) Visual observation of plant material performance: Indicate damage inflicted by abnormal waterflow, debris accumulating, overall judgment of how plants have performed and infiltration of other growth.	Inspection, and/or following highwater coordinated with ERS.
e. Environment (Environmental Resources Section, Coordinated with Civil Design)	
(1) Preconstruction Habitat Evaluation	Preconstruction.
(a) Vegetation and Wildlife Monitoring	Construction (see pages 4 and 5 for details).
(b) Evaluation Report	Following construction.
(2) Postconstruction Habitat Evaluation	
(a) Vegetation and Wildlife Monitoring and Habitat Evaluation	Monthly sampling and analysis yearly evaluation report (see pages 4 and 5).
(b) Comparative Evaluation	Once, at the end of monitoring period.

INCLOSURE 4 (SHEET 2 OF 5)

WORK ITEM	FREQUENCY
f. Photography	
(1) Ground level and aerial still moving photograph (Project Manager).	Prior to, during, and following construction.
(2) Ground level photos durings inspections (ERS, Program Manager)	Following construction.
g. Maintenance	
Describe and record cost of any required maintenance (all).	As required.
h. Report of Observations	
(1) Written memo required following all investigations or inspections. Memos to be provided Project Manager with copy to members of monitoring team.	Following field inspection visits and any office studies or evaluations.

GREEN RIVER STREAMBANK PROTECTION DEMONSTRATION PROJECT
KENT, WASHINGTON

SUBJECT: Detail Plan for Habitat and Vegetation Evaluation

1. Preconstruction Habitat Evaluation.

a. Vegetation Monitoring

(1) Sampling Methods. Preconstruction monitoring of vegetation used qualitative methods only. Sampling consisted of a cumulative species list along a north-south transect with notations of site conditions and estimates of coverage where possible. Visual observations of aquatic habitat were made. This sampling procedure will not affect subsequent sampling or statistical analysis, as comparison will be made primarily between treatments for effectiveness of erosion control rather than as a comparison of conditions before and after construction.

(2) Analysis. Vegetation will be identified in the project area and keyed to species whenever possible. Data will be used to determine species diversity and dominance (percent cover).

b. Wildlife Monitoring

(1) Sampling Methods. During each field visit to the project area, bird usage will be monitored using line transects, and mammal usage will be monitored by visual observations.

(2) Analysis. A wildlife species list for the project area will be developed for use in determining species diversity. Estimates of bird population densities will be made from line transect data. This information will be recorded for each field visit.

c. Habitat Evaluation. Based on analysis of field data and knowledge of the wildlife values of vegetation species present, an evaluation of overall project area terrestrial habitat quality will be made, with specifics relating to each of the six 300-foot project sections. The aquatic habitat quality will be generally described. The information per project section will serve as controls for the postconstruction monitoring program.

2. Postconstruction Habitat Evaluation.

a. Vegetation Monitoring.

(1) Sampling Methods. For each field visit (one per month) general physical features of the project area will be described. A quantitative analysis of vegetation will be made using permanent test plots for each of the five subplots within the six 300-foot project

sections shown on planting drawings. The size and number of permanent plots established will be determined after project construction. Visual observations of aquatic habitat will also be made.

(2) Analysis. Vegetation will be identified in the project area and keyed to species whenever possible. Data will be used to determine species diversity, density, size of individual plants, and species dominance (percent cover). Measurements will be recorded for each field visit. Records will be kept of percent survival of planted species and revegetation by native species for each plot.

b. Wildlife Monitoring. The postconstruction monitoring program will generally consist of the same work items as the preconstruction program. Bird line transects and visual observations of mammals will be accomplished on a monthly basis. At the same time, any noticeable changes in aquatic habitat characteristics will be recorded and described.

c. Habitat Evaluation. A complete evaluation of field data for the postconstruction monitoring program will be accomplished on a yearly basis and will include an assessment of habitat quality, vegetative planting success, and wildlife usage.

d. Comparative Evaluation. At the end of the postconstruction monitoring program, an evaluation comparing pre and postconstruction vegetation and habitat quality will be made. All field data will be evaluated using a statistical analysis of variance for comparing preconstruction conditions with postconstruction conditions and for assessment of differences between and within project sections over time. Any significant differences will be noted and evaluated in terms of vegetation and habitat quality changes and the success (from a habitat value standpoint) of the various erosion control methods. Recommendations for future riverbank construction activities will be made.

3. Reporting Procedures.

a. Field data will be recorded on sheets indicating locations. Records will be maintained for the entire monitoring period.

b. Photographic documentation will be maintained for each field visit.

c. Evaluations will be in short and concise narrative report format.

4. Field Team. The field team will consist of one wildlife biologist and one botanist/ecologist. Overall project supervision and preparation of evaluation reports will be the responsibility of a senior environmental planner.

5. Coordination. The monitoring program will be coordinated with Federal and state resource agencies. Interested agencies will be invited to accompany the field team on one inspection of the project area per year.

KANSAS RIVER NEAR
EUDORA, KANSAS

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

KANSAS RIVER AT FALL LEAF BOTTOMS DRAINAGE DISTRICT
NEAR EUDORA, KANSAS
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Fall Leaf Bottoms Drainage District, Kansas River, Mile 43 to 44, Eudora, Kansas. Figure 1 shows location map.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem, the types of bank protection used, and a performance evaluation of a demonstration project on the Kansas River, Kansas, constructed and monitored by the Kansas City District.

4. Problem Resume. The left bank at this location has been eroding continuously for several years. The rate varied with climatic conditions. The high bank varies in height from 19 to 30 feet above the streambed and is subject to occasional inundation at the lower elevations. The land is farmland and is valued at $1,500 to $2,000 per acre (1980 prices).

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The Kansas River basin drains the northern half of Kansas, the southern part of Nebraska, and the northeastern part of Colorado and comprises an area of 60,060 square miles, or roughly 11.4 percent of the Missouri River watershed. Approximately 60 percent of the Kansas River basin lies within Kansas. The basin is about 480 miles long in an east-west direction by about 140 miles wide, and lies entirely

within the Interior Plains Region. Valleys of the main stem and the lower reaches of principal tributaries are wide, alluvial bottom lands. Only in the headwater regions are the streams deeply entrenched. From the mouth of the river at Kansas City, elevation 750 feet above mean sea level (m.s.l.), the general land surface rises to elevation 5,500 feet, m.s.l., at the extreme western edge of the watershed.

The Kansas River is formed by the confluence of the Republican and Smoky Hill Rivers near Junction City, Kansas. It then flows eastward for about 169 miles to Kansas City, where it joins the Missouri River at river mile 365.4 (1960 mileage). Principal tributaries of the Kansas River proper include the Big Blue and Delaware Rivers in the easterly portion of the drainage area. The Republican River rises in northeastern Colorado and flows eastward for about 420 miles across Nebraska and Kansas to Junction City. The Smoky Hill River rises in eastern Colorado and flows eastward for 500 miles to its confluence with the Republican River. The Big Blue River rises in southern Nebraska and flows generally southward for 287 miles to its junction with the Kansas River at Manhattan, Kansas. The Delaware River rises in northeastern Kansas and flows southward 100 miles to join the Kansas River at mile 63.3 near Perry, Kansas. The river slopes 1.7 feet per mile in the study area.

b. Geology. The study area is in the physiographic Central Lowlands Province which is divided into the Dissected Till Plains Section and the Osage Plains Section. The land surface, which is generally flat and low-lying, is characterized by flood plain and meander belt topography. With few exceptions, overburden in the flood plains of the Kansas River and its tributaries is alluvial. The alluvium consists of stream laid deposits of clay, silt, sand and gravel; the character and proportions of which differ from one place to another. The finer materials commonly occur closer to the ground surface and the coarser materials at greater depth. Also within the flood plains of the study area,

three or more Pleistocene terraces can be identified. These terraces range from a few feet to over 100 feet above the valley floors and represent climatic and associated sea level changes in different stages of the Pleistocene.

Glacial till occurs on the uplands from the northern part of the Osage Plains section and covers the entire Dissected Till Plains section. Generally, this till is made up of unsorted mixtures of clay, silt, sand, gravel, cobbles, and boulders, and a large part is overlain by eolian (loess) material. The glacial drift sheet responsible for the deposition of this till has been differentially eroded and therefore varies considerably in thickness. Unconsolidated deposits of Pleistocene age form the surfacial material in the study area. This unconsolidated material is glacial drift mantled by eolian deposits.

c. Hydrologic Characteristics. At the study area, the variations from normal climatic conditions from season to season and from year to year can be extreme. The climatic history of the area includes intense and prolonged rainfall during some years and severe droughts in others without apparent fixed cyclic pattern. The average annual precipitation for the study area varies from 23 to 36 inches. The heaviest periods of precipitation are generally during the spring and early summer months. Precipitation during the late summer and fall months is usually of the short duration thunderstorm type with small centers of high intensity, although widespread general rains occasionally occur. Winter precipitation amounts are, in general, considerably less than for other seasons of the year. The maximum recorded discharge of 510,000 c.f.s. occurred during the period of 10 to 20 July 1951. At least 20 major floods have been experienced on the Kansas River since 1900. However, a system of flood control lakes on all of the major tributaries now controls 84 percent of the basin and the possibility of another major flood is greatly reduced. Releases of water from flood control storage can result in long duration of half to bank full flows.

d. Channel Conditions. The Kansas River through the study area is typical of a mature major river in the geographical region. Its valley is quite wide and crossed by gentle meanders. The relatively wide channel is occupied by a deep sand bed and is experiencing degradation in certain areas. Major channel alinement changes have not generally occurred except during major floods.

e. Environmental Considerations. The study area is predominately agricultural land with only a small part of the bankline covered with trees. The project will stabilize a raw, actively eroding bank and will not destroy or alter terrestial wildlife habitat. Stream turbidity will be reduced in the study area and will have a beneficial effect on the aquatic habitat.

6. Demonstration Site.

a. Hydrologic Characteristics. The hydrologic characteristics are as previously stated. A USGS gaging station is located at DeSoto, Kansas, approximately 12 river miles downstream of the project site. Figure 2 is a table summarizing the discharges since completion of construction.

b. Hydraulic Characteristics. Average flow velocities at the study site range from 2 ft/s at a discharge of 2,000 c.f.s., to 4.5 ft/s at a 500-year flood level (428,000 c.f.s.).

c. Riverbank Description.

(1) Bank Materials. Materials composing the banks and valley floor of the Kansas River consist of silts and lean clay with sand lenses. Test borings at the study site are summarized on Figure 3.

(2) Normal Bank Vegetation. Vegetation on the sloping bank in the study area was, for all practical purposes, nonexistent. Vegetation

on the high bank consists of most agricultural crops, with a small area of native grasses, willows, and cottonwoods.

(3) Bank Erosion Tendencies. The test site, Figure 4, had been eroding at a rate of approximately 30 to 35 feet per year over the last 6 years. In the past 30 years, the overall average rate has been approximately 60 feet per year, with most of the change coming during the flood of 1951.

III. DESIGN AND CONSTRUCTION

7. General. The high bank and fluctuating flows at the Fall Leaf site, coupled with the easily eroded soil, dictated some form of heavy stabilization work.

8. Basis for Design. The primary reason for electing to construct a windrow revetment with varying application rates, Figure 5, and a section of stone reinforced toe was ease of construction of a relatively massive stabilization project. In addition, this type of stabilization work had not previously been used along the Kansas River. A test area was needed to determine if the rapid erosion rate could be controlled by allowing the river to establish the slope. In addition, the effectiveness of a windrow revetment needed to be determined in an area subject to a wide variation in discharges. The reinforced toe section was designed to stabilize the bank adjacent to the wooded area with a minimum of environmental disturbance.

9. Construction Details. The windrow revetment was constructed by end dumping the stone along the structure alinement, with final shaping by a dozer. The alinement, rates of application, and typical cross sections are shown in Figure 6. The quantities were carefully controlled during construction so that the design rates were met. The reinforced toe section was constructed by using a dragline with a rock bucket. The specifications called for quarry-run stone reasonably well graded, free from

overburden, spoil, shale, and unsuitable material. The amount of dirt and fines less than 1/2-inch maximum cross section, accumulated from interledge layers or from blasting or handling operations, shall not exceed 5 percent by weight. The maximum weight of any piece of stone shall be limited to 1,000 pounds.

10. Cost. The total cost of the windrow revetment was $79,135 or approximately $43 per bank-foot. Total cost of the reinforced toe was $23,132 or approximately $58 per bank-foot. Construction of a standard revetment would have cost an estimated $385,000 or $170 per bank-foot.

IV. PERFORMANCE OF PROTECTION

11. Monitoring Program. Construction was completed 18 May 1979 and the site has been monitored for the past 2 years. Detailed surveys were made prior to initiation of construction and immediately following construction. Another survey was made 1 year after completion to determine the rate of change. Figures 7 and 8 illustrate the results of these surveys. The site has been visually inspected following each major change in discharge and during periods of high flows. Ground level photographs were taken during each inspection trip and Figures 9 through 13 show in chronological order the changes in site conditions.

12. Evaluation of Protection Performance. As of July 1981, stone from approximately 750 linear feet of the windrow revetment had been displaced. The first stone to be displaced was in October 1979 in the area between stations 16+00 to 17+00. By December 1980, this area had expanded to 450 feet between stations 15+00 and 19+50. Displacement down the bank and into the river had also occurred in three other areas totaling approximately 300 linear feet. In the area between stations 15+00 and 19+50, one-third to one-half of the stone has migrated down the bank and is armoring the bank 7 to 8 feet above mean low water. Weeds and small cottonwoods have started to grow on the slope. In the other areas where displacement has begun, the amount of stone displaced is small. In the

area along the bank above the reinforced toe, vegetation is becoming reestablished. Figure 6 indicates the areas where stone is being displaced.

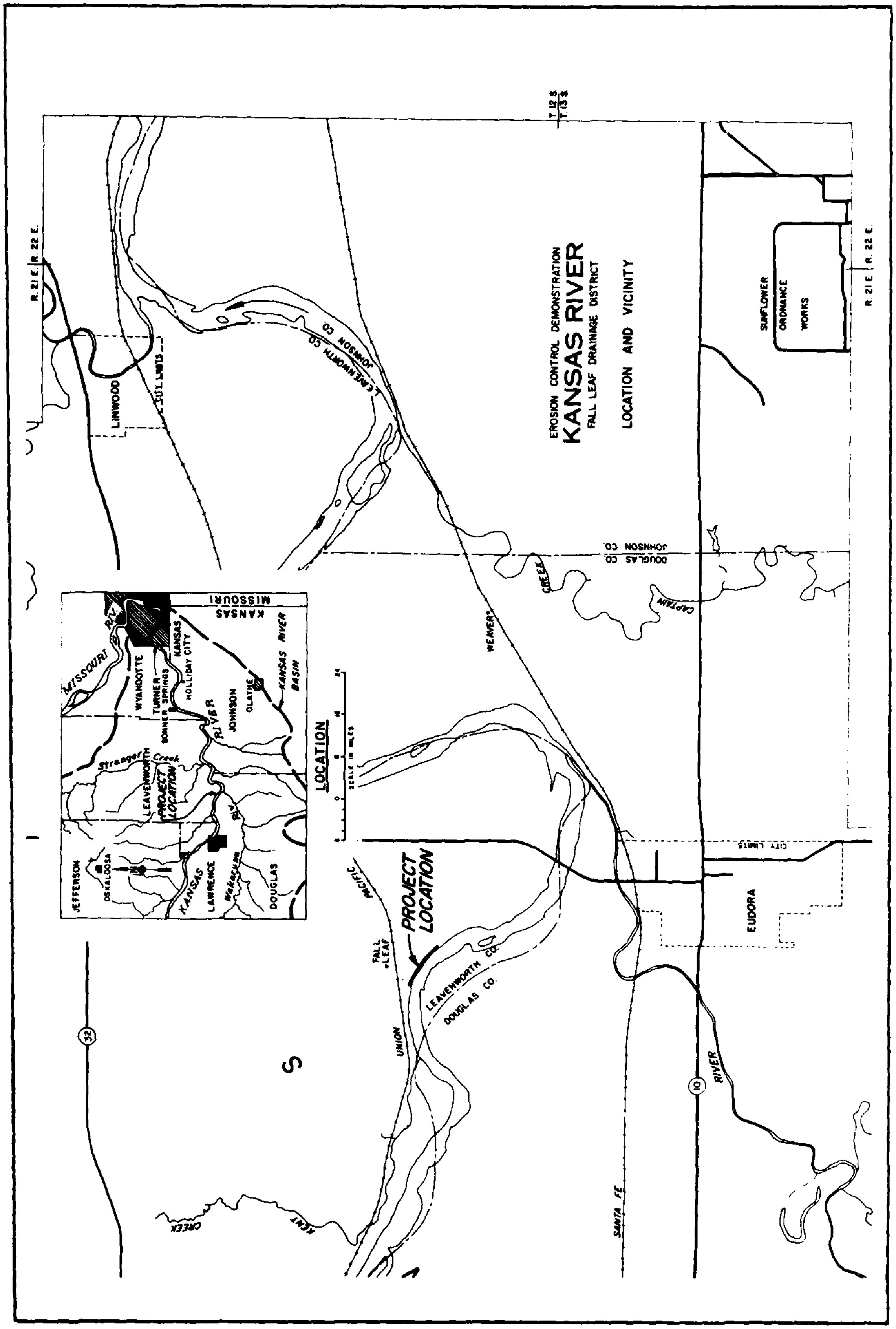

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT
LOCATION AND VICINITY
SUNFLOWER
ORDNANCE
WORKS
LINWOOD
EUDORA
PROJECT LOCATION
LEAVENWORTH CO.
DOUGLAS CO.
JOHNSON CO.
UNION
PACIFIC
FALL LEAF
SANTA FE
RIVER
WEAVER
CAPTAIN
CREEK
KENT
CITY LIMITS
LOCATION
MISSOURI
KANSAS
JEFFERSON
OSKALOOSA
LEAVENWORTH
WYANDOTTE
TURNER
BONNER SPRINGS
HOLLIDAY CITY
JOHNSON
OLATHE
LAWRENCE
DOUGLAS
KANSAS RIVER BASIN
Stranger Creek
R. 21 E. | R. 22 E.
T. 12 S.
T. 13 S.

FIGURE 1

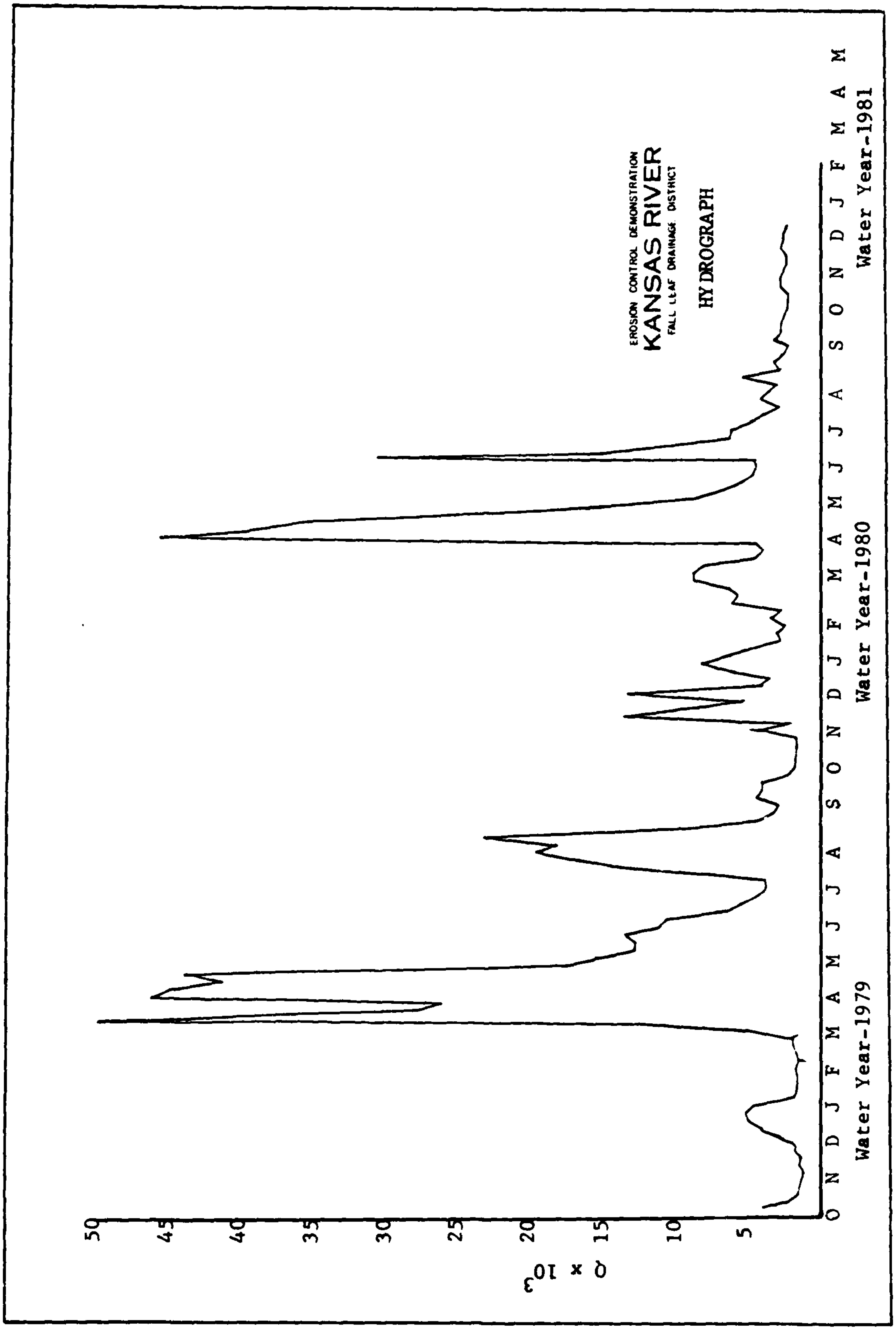

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT
HYDROGRAPH
Q x 10^3
50
45
40
35
30
25
20
15
10
5
O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M
Water Year-1979
Water Year-1980
Water Year-1981

FIGURE 2

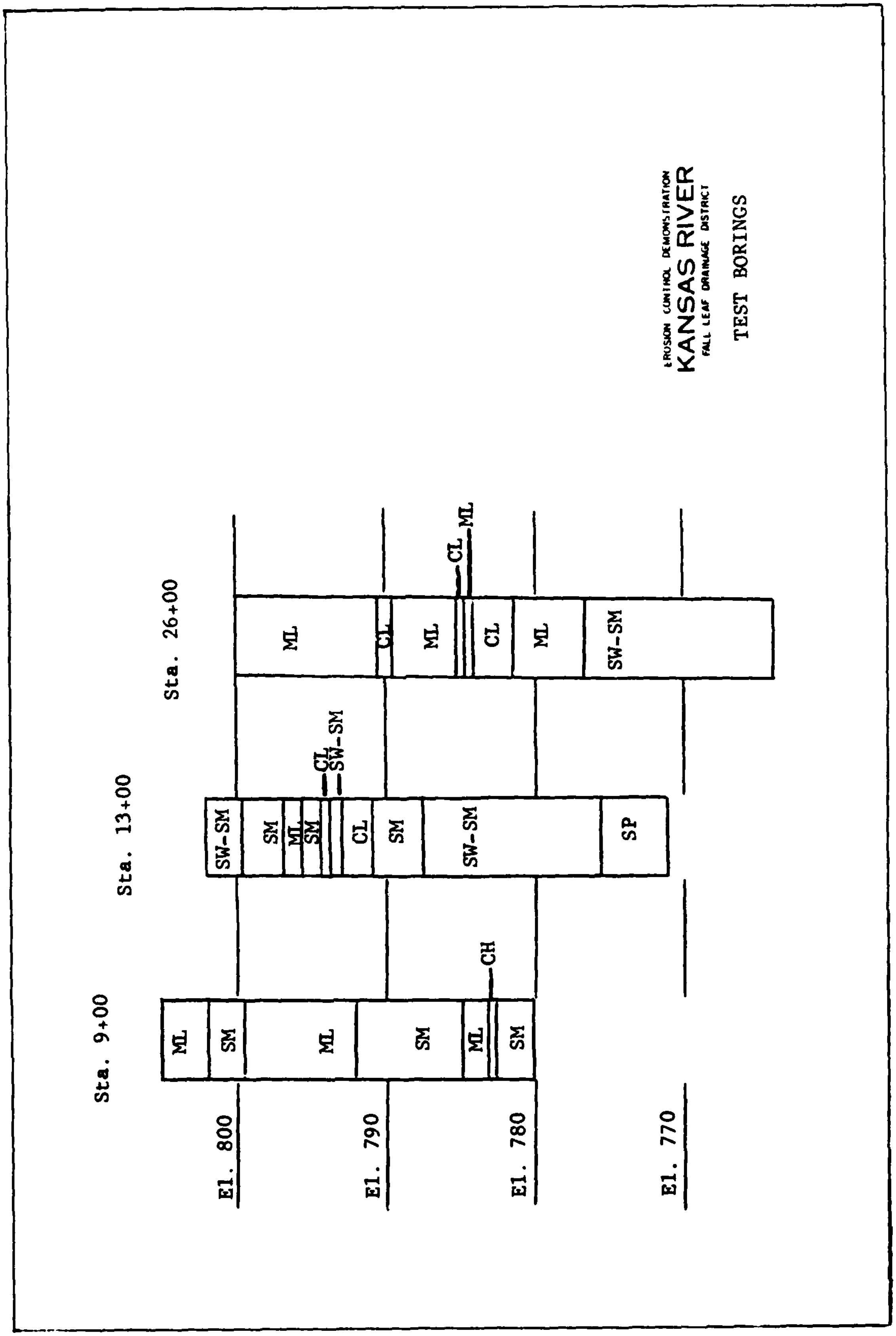

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT
TEST BORINGS
Sta. 9+00
Sta. 13+00
Sta. 26+00
El. 800
El. 790
El. 780
El. 770
ML
SM
ML
SM
ML
CH
SM
SW-SM
SM
ML
SM
CL
SW-SM
CL
SM
SW-SM
SP
ML
CL
ML
CL
ML
CL
ML
SW-SM

FIGURE 3

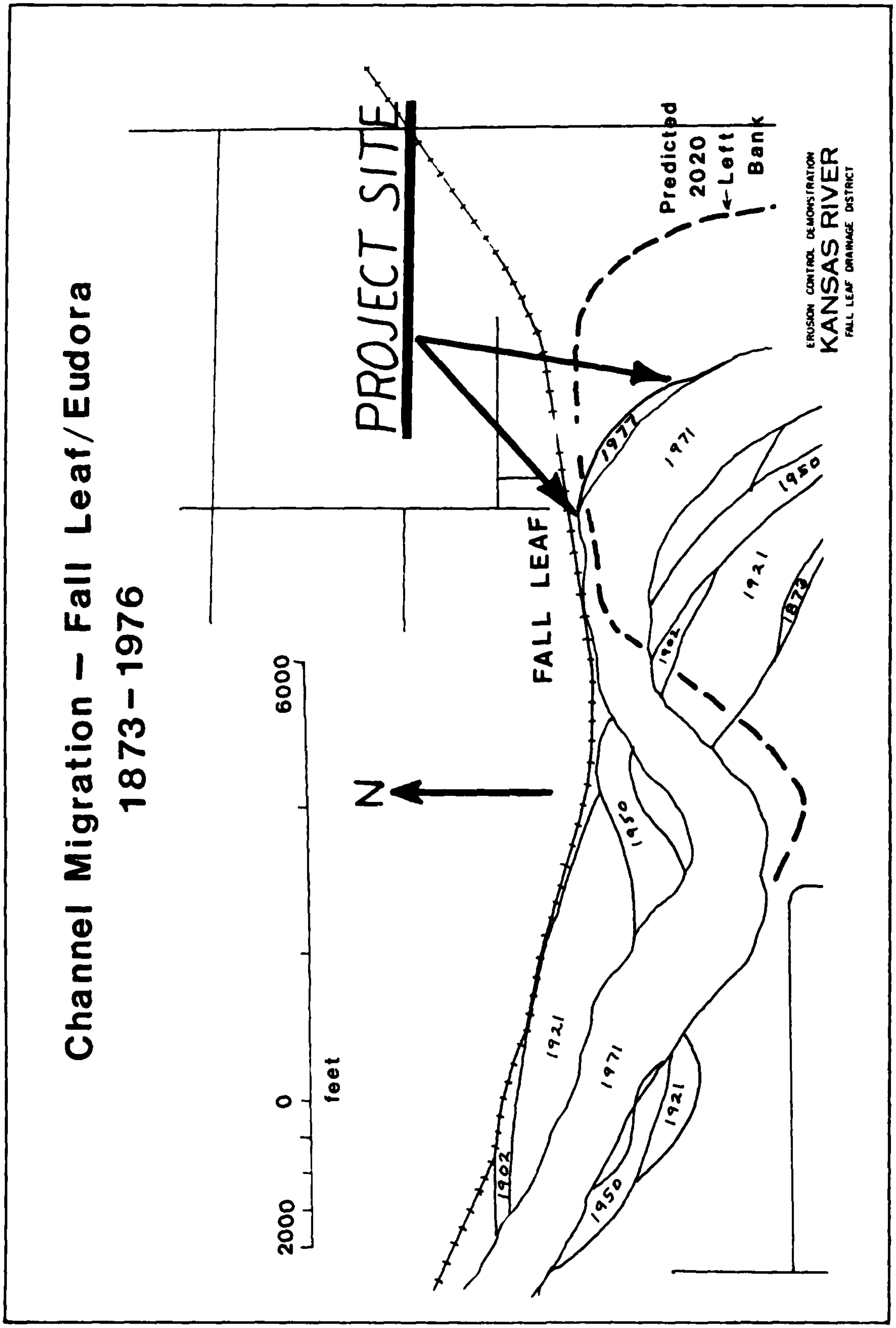

Channel Migration — Fall Leaf/Eudora
1873 - 1976
PROJECT SITE
Predicted
2020
←Left
Bank
EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT
FALL LEAF
N
6000
0
2000
feet
1977
1971
1950
1921
1902
1873

FIGURE 4

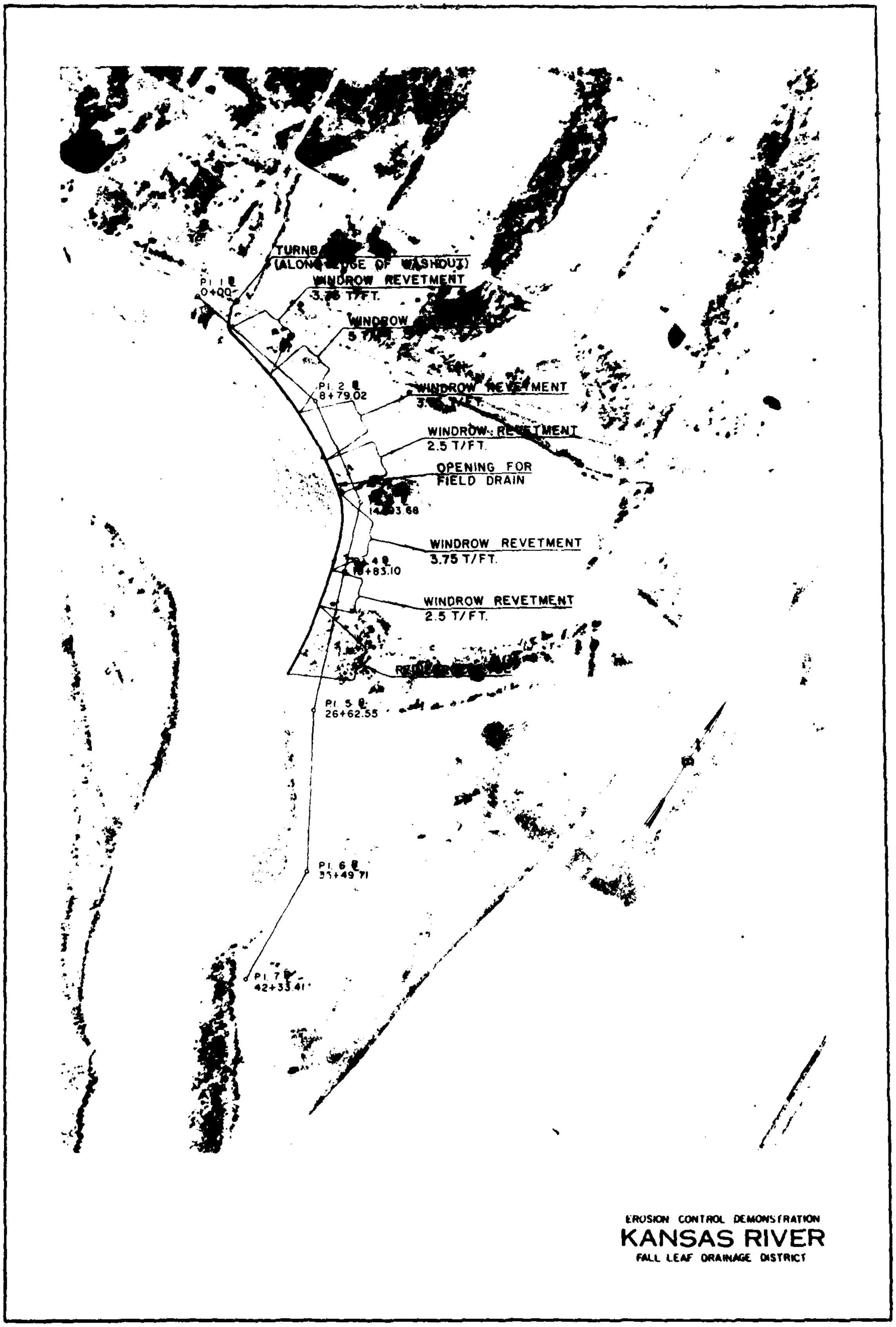

P.I. 1
0+00
WINDROW REVETMENT
P.I. 2
8+79.02
WINDROW REVETMENT
2.5 T/FT.
OPENING FOR
FIELD DRAIN
WINDROW REVETMENT
3.75 T/FT.
WINDROW REVETMENT
2.5 T/FT.
P.I. 5
26+62.55
P.I. 7
42+33.41
EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 5

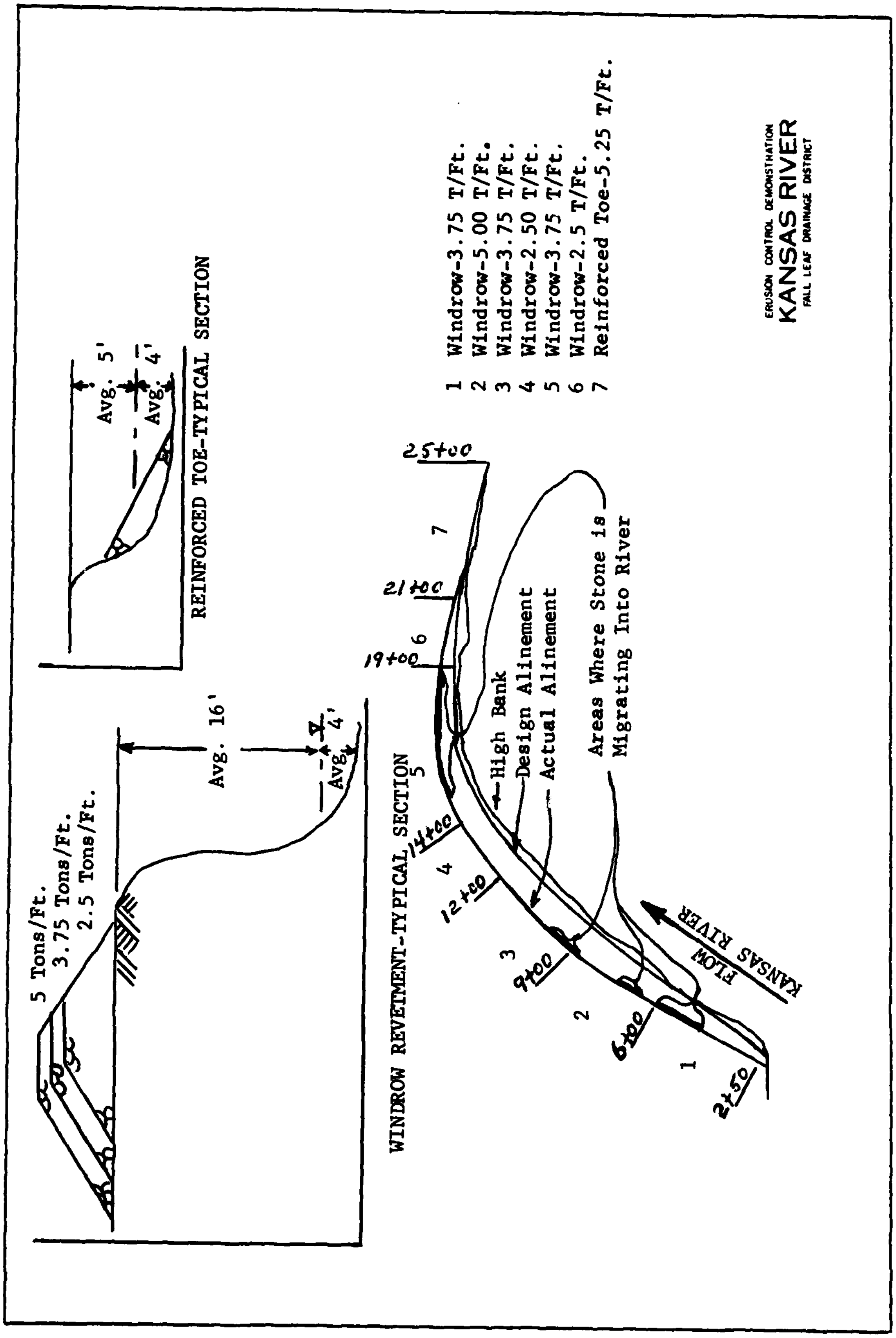

5 Tons/Ft.
3.75 Tons/Ft.
2.5 Tons/Ft.
Avg. 16'
Avg. 4'
WINDROW REVETMENT-TYPICAL SECTION
Avg. 5'
Avg. 4'
REINFORCED TOE-TYPICAL SECTION
2+50
6+00
9+00
12+00
14+00
19+00
21+00
25+00
1
2
3
4
5
6
7
High Bank
Design Alinement
Actual Alinement
Areas Where Stone is
Migrating Into River
FLOW
KANSAS RIVER
1 Windrow-3.75 T/Ft.
2 Windrow-5.00 T/Ft.
3 Windrow-3.75 T/Ft.
4 Windrow-2.50 T/Ft.
5 Windrow-3.75 T/Ft.
6 Windrow-2.5 T/Ft.
7 Reinforced Toe-5.25 T/Ft.
EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 6

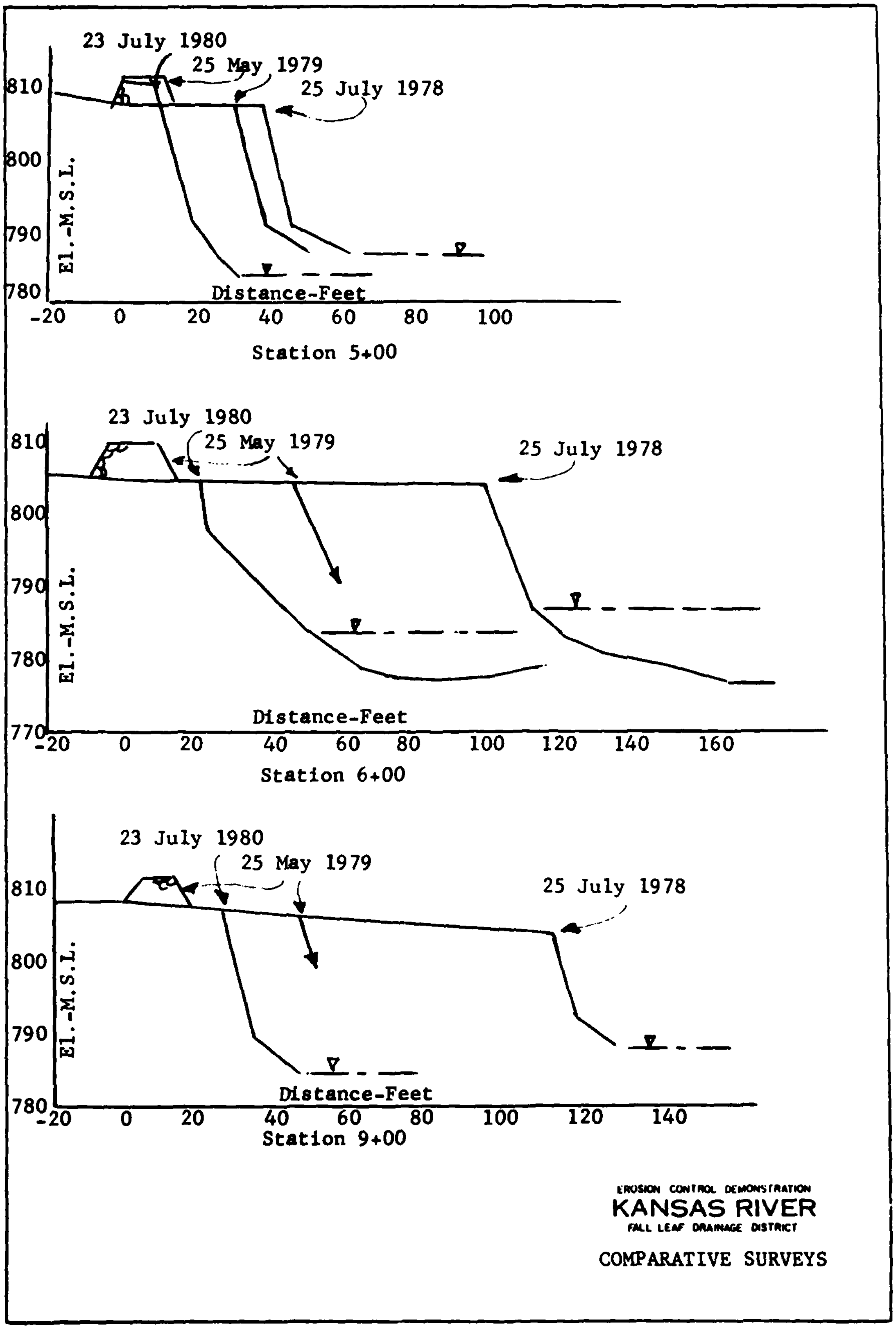
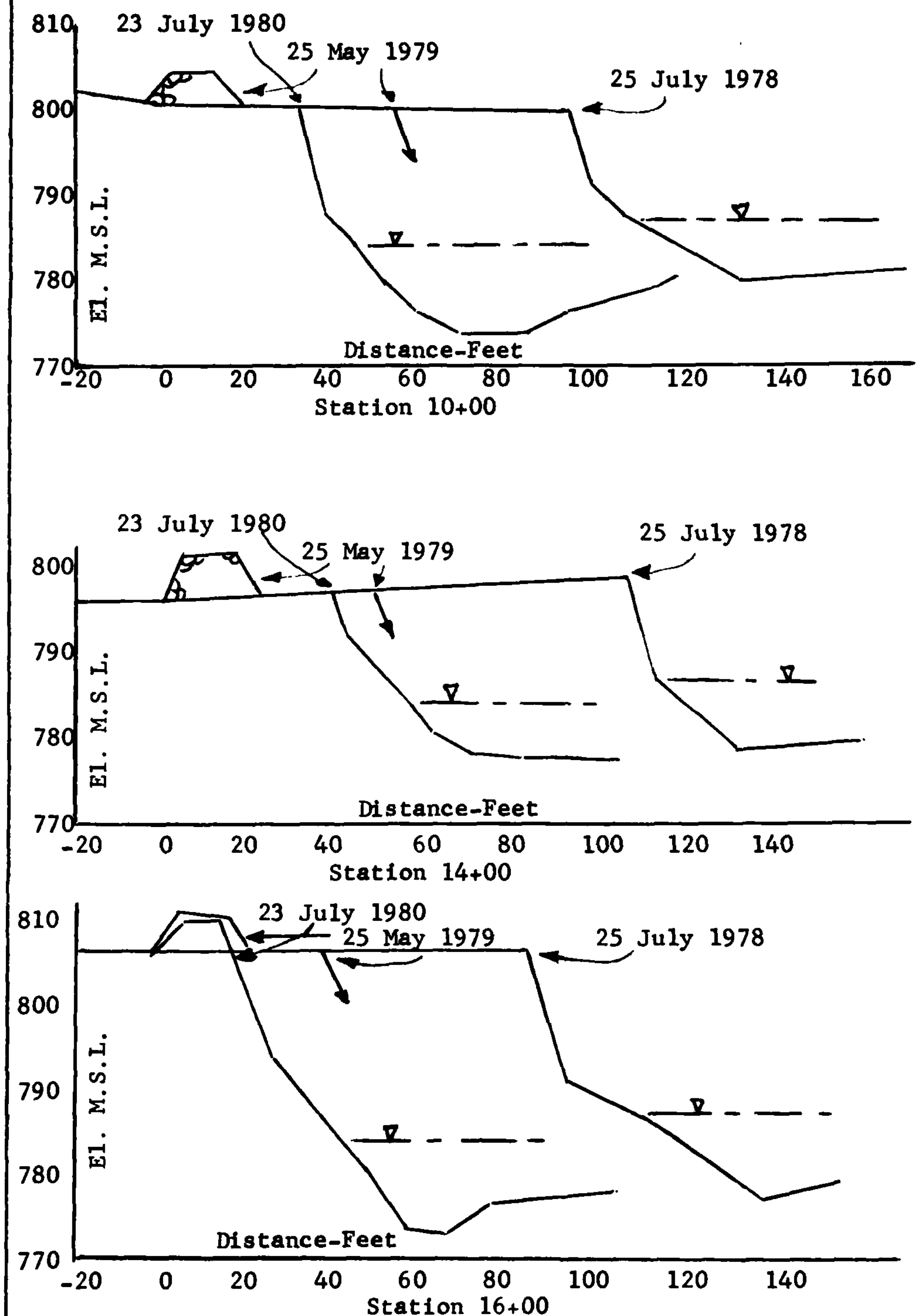

23 July 1980
25 May 1979
25 July 1978
810
800
790
780
El.-M.S.L.
Distance-Feet
-20
0
20
40
60
80
100
Station 5+00
23 July 1980
25 May 1979
25 July 1978
810
800
790
780
770
El.-M.S.L.
Distance-Feet
-20
0
20
40
60
80
100
120
140
160
Station 6+00
23 July 1980
25 May 1979
25 July 1978
810
800
790
780
El.-M.S.L.
Distance-Feet
-20
0
20
40
60
80
100
120
140
Station 9+00
EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT
COMPARATIVE SURVEYS

FIGURE 7



EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

COMPARATIVE SURVEYS

FIGURE 8

Station 5+00 Before Construction

Station 5+00 After Construction

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 9

Station 5+00 18 Months After Completion

Station 8+80 Before Construction

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

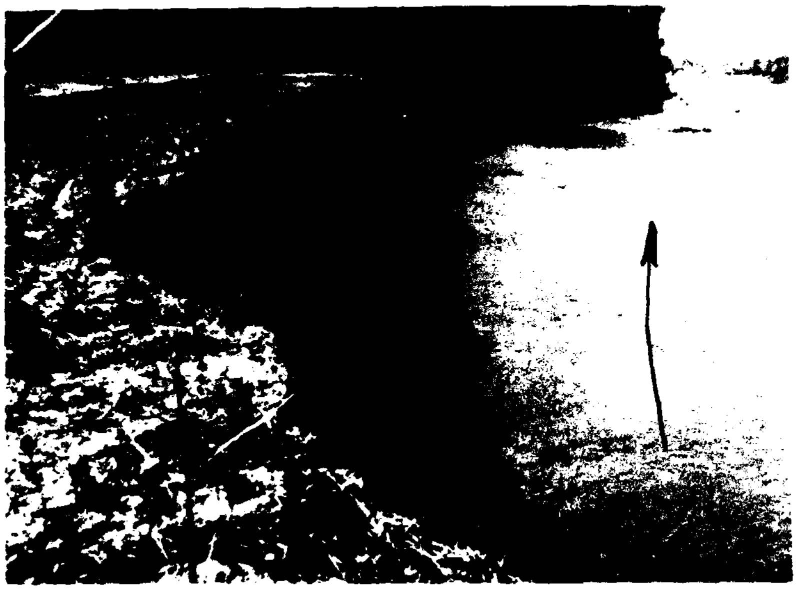
FIGURE 10

Station 9+00 After Construction

Station 9+50 18 Months After Completion

ERUSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 11



Station 15+00 Before Construction

Station 15+00 After Construction

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 12

Station 15+00 18 Months After Completion

EROSION CONTROL DEMONSTRATION
KANSAS RIVER
FALL LEAF DRAINAGE DISTRICT

FIGURE 13

KANAWHA RIVER AT
SOUTH CHARLESTON, WEST VIRGINIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

KANAWHA RIVER AT SOUTH CHARLESTON, WEST VIRGINIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

A. Project Name and Location. South Charleston, Kanawha County, West Virginia, along the left descending bank at and for a 1600 foot reach above Kanawha River 52.5 with State Coordinate System references of N 494,619.8 and E 1,791,071.5. Exhibit No. I-1 shows the project location.

B. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, P.L. 93-251.

C. Purpose and Scope. This report describes a bank failure and erosion condition, types of treatments used, and a performance evaluation of a demonstration project on the Kanawha River designed and monitored by the Huntington District.

D. Problem Resume. The left bank of the Kanawha River upstream of Dunbar Highway Bridge and downstream of the Spring Hill Light was subject to active failure and erosion with resulting top of slope retreat and variable encroachment on a road system and homesites.

II. HISTORICAL DESCRIPTION

A. Stream Description, General.

1. Topography. The Kanawha River at the demonstration site drains an area including portions of Pennsylvania, Virginia, West Virginia, and North Carolina. Major tributaries are the New Gauley and Elk Rivers. The topography of the basin is characterized by mature development of the drainage systems within the Kanawha Physiographic Section of the

Appalachian Plateau Province. The Kanawha River is formed by the confluence of the Gauley and New Rivers at Gauley Bridge, West Virginia. From Falls View the Kanawha River descends about 50 feet along a course of 40 miles to the demonstration site. Relief at the site is approximately 800 feet from the river to the top of the surrounding valley walls. The river flows in a westerly direction at the demonstration site following a course with broad curves varying from 60 degrees to 150 degrees and radii of about 5 miles. The natural stream gradient in this area is about 0.8 foot per mile. The valley floor is about one mile in width. Stream bank heights are about 30 feet above normal pool at the South Charleston Project. The demonstration site is located along a wide alluvium terrace near the Coal River confluence, approximately one-half mile downriver from Wilson Island, and extends upstream 2 miles on the outside of a shallow Kanawha River bend.

2. Geology. The Kanawha River throughout its course has become entrenched in sedimentary strata of Pennsylvanian Age. These strata are made up of interbedded sandstones, siltstones, clay, shales, limestones, and coals. The bedrock valley of the Kanawha River contains recent alluvium. In the portion of the Kanawha Valley within the project reach, these materials are often sand overlain by silty sand and clayey silt. Since the last glacial episode, the Kanawha River has cut through and laterally into these materials with river terraces remaining as erosional remnants at various elevations in these alluvium and most often by point bar accretion with the forming of a defined flood plain. In the study area, the Kanawha River is underlain to depths of approximately 35 feet.

The Kanawha River channel has changed location by lateral cutting and filling within alluvium which fills the bedrock valley. The channel is, however, often bedrock controlled in its location to one side of the Pleistocene defined valley. One bank often consists of flood plain deposits and the other of rock outcrops or colluvial soil which has accumulated through slope materials weathering, creep, and landslides. The colluvial soils are generally stiff silty clays with angular rock

fragments and little or no layering. These soils tend to be somewhat resistant to river related erosion.

3. Locality, Development, and Occupation. The Kanawha River valley in the vicinity of the demonstration site has a diverse and urban industrial land use. Over the past one hundred years most of these broad agricultural bottoms have been acquired for industrial development. Within the Winfield navigation pool the river valley contains several small towns including Red House, Winfield, Nitro, St. Albans, Institute, Dunbar, S. Charleston, Charleston, and Marmet. Local industries include coal mining, quarries, steel, alloys, and chemical production, electric power generation, and a variety of light manufacturing. The river is paralleled by railroads and highways located approximate to both banks.

The Kanawha River drainage has been an important transportation system since prehistory and has been improved for navigation with the construction of a system of locks and movable dams beginning in 1875. The system of ten locks and dam structures was completed in 1898. The ten structure low-lift system was replaced during 1931 to 1937 period with four high-lift structures. Hydroelectric power plants are operated at each of the Kanawha River dams. In 1893, Lock and Dam 7 was put into operation 8 miles downstream of the demonstration site.

These early dams incorporated a navigable pass to provide a channel for open river navigation during periods of high flow. A series of wickets, heavy timber shutters, were raised to impound water as needed to maintain a navigation pool. When not required, the wickets would lie flat at such a depth to offer no obstructions to free navigation through the pass. In 1937, Winfield Locks and Dam went into full operation and Lock and Dam 7 was removed.

4. Hydrologic Characteristics. The climate of the site reach and Kanawha River is continental with marked contrasts and average annual temperatures of about 54°F and an average annual precipitation of 44" rainfall. The period from 1970 through 1976 was determined to be wetter than average. 1979 was a wet year while 1980 was average and 1981 (from

1 October 1980 to February 1981) was dryer than average. Ice occurs on all rivers and streams in the basin with the Kanawha River being froze over for significant reaches in the winters of 1976-77 and 1977-78. Major floods affecting the Kanawha River occurred in January 1937, March 1945, and March 1964.

5. Existing Channel Conditions. The sinuosity of the channel was described in paragraph II.A.1. The channel location and width-depth relationships have been relatively stable within historical time.

6. Environmental Considerations. Active farming and old fields are frequently encounterd within the Kanawha River floodplain area. The steep hillsides adjacent to the valley floor are primarily undeveloped and consist of second growth woodlands. Within the floodplain, vegetation associated with farming and frequent site disturbance prevails. Along the river bottom land, silver maple, sycamore, and willow occur. On the hillsides and in areas of bank and slope above ordinary high water, oaks, beech, red maple, ash, black cherry and walnut exist. Nails, spikes, eye bolts, cables and physical damage to trees from river traffic are also evident.

Fish in the project area include channel catfish, carp, white bass, pumpkinseed, bluegill, white crappie, shiners, perch, and gizzard shad. Excellent warm-water fisheries have developed at or near the mouths of several of the tributary streams. Area wildlife resources include mourning doves, bobwhite quail, and cottontail rabbits in the approximate agricultural areas, while ruffed grouse and squirrels inhabit the uplands. Whitetailed deer are present in the adjacent uplands and also range into the valley. The Kanawha River also provides resting and feeding opportunities for several species of migratory waterfowl. Muskrat, raccoon, and fox are some of the fur animals in the area.

This reach of the river, as with the entire Kanawha is exposed to various types of pollution which tend to affect aquatic life and generally detract from the aesthetic value of the river. Organic matter, chemicals, sediment, and colloidal material contribute to

relatively poor water quality, with seasonal variations also resulting from changes in flow and temperature.

An environmental assessment was prepared in accordance with Section 404(b) of Public Law 92-500. Impacts of construction at the site were addressed in the assessment and the effects of each bank protection scheme were considered, as were total project effects. Modifications to the riverbank during construction will cause localized and minor adverse ecological effects including degraded water quality. Construction of this project was determined to result in net beneficial environmental effects within the riverbank area.

B. Demonstration Project.

1. Hydrologic Characteristics. Channel cross sections have been determined to be generally consistent as to width-depth relationships and prehistorical features. The river channel in the immediate vicinity of the demonstration project has been subject to sand and gravel dredging. Ice formation in the project area becomes significant only during unusually severe winters. Ice movement is not a factor in bank erosion at the site.

2. Hydraulic Characteristics. Average velocities have been determined for frequently occurring excessive flow events as approximately 5 feet per second within the Kanawha River channel. Waves have been observed under various wind and traffic conditions. Maximum wave height was approximately two feet. The minimum pool for navigation use is retained at elevation 566 by the Winfield Locks and Dam 21 miles downstream. The dam gates are raised to pass high flows, so that the influence of the dam on the river decreases with increasing flow. The influence of these navigation dams during excessive flow events is insignificant. Prior to the completion of the Winfield Locks and Dam in 1937, the Ohio River at the demonstration site was maintained at minimum pool elevation 554.65 by Lock and Dam 7. The stage hydrographs for this project site are referenced as Exhibit I-8.

3. Riverbank Description. The riverbank at the demonstration site

is approximate to a bridge and includes extensive fill and is characterized by moderate height and variable slopes. The bank is composed of fine grained alluvium deposited as point bars and during overbank event falling stages. These interbedded and interlensing sediments include silty clays overlying silty sands and sands. In the project reach of bank, dump debris and random filling are frequently encountered. A typical geotechnical cross section of the project site is referenced as Exhibit I-7.

The area immediately landward of the top of bank has low relief and is a paved street and homesites. Kanawha Street travel lane is at top of bank.

Most failure of banks at this site occurs during flood events and as the river returns to near normal stages. A frequently encountered failure sequence in these alluvium includes internal erosion of sand and silty sand by groundwater flowing out of the riverbank, referenced as "piping" with resultant weakening of overlying soils. The bank then fails by drawdown related slumpages and slabbing as the river falls from flood stages with current related erosion of in situ soils and the failed debris.

The District has been aware of bank failure and erosion at this site since historical times and inclusive of the channelization period. Photographs submitted as Exhibit Nos. II-1 to II-8 indicate the condition of the bank prior to construction, during construction, and after construction of the demonstration project.

III. DESIGN AND CONSTRUCTION

A. General. The South Charleston, West Virginia, site was used to evaluate four different schemes of bank protection along a reach of quite variable upper bank topography.

B. Basis for Design. Treatments were intended to generally address normal pool land-water interface contacts with upslope conditions being

considered at some test sections. These materials were not intended to protect against mechanisms which are most significant and occur during major storms and floods. The structural features included a tire mat, soil-cement buttress, floating tire breakwater, and waste stone buttress. Placement of these materials was somewhat labor intensive but were included as being feasible for use by small property owners with limited financial resources. Vegetation covers included grasses and pussy willow and red osier dogwood cuttings, and red maples.

C. Construction Details. A tire mat, soil-cement buttress, floating tire breakwater and waste stone buttress were placed as indicated on Exhibit Nos. I-2 thru I-5. Riprap and waste rock and concrete rubble were placed at treatment limits. Project location, limits, and extents of each treatment are shown on Exhibit No. I-1. Treatments were completed in 1977 and included the following materials:

Scheme A - Lower reach of regraded upper bank protected with a tire mat.

Scheme B - Toe of upper bank buttressed and protected with soil cement.

Scheme C - Off bank placement of a floating tire breakwater.

Scheme D - Upper bank toe protection using fine (5-in. top size) waste stone.

D. Costs. These treatments were constructed by December 1979. Total costs including monies to construct was $545,450. A cost breakdown for each test section, showing cost per lineal foot and cost per square foot, is included as Exhibit No. I-6.

IV. PERFORMANCE OF PROTECTION

A. Monitoring Program. Monitoring included reach of river and site specific reconnaissance, photography, mapping, sampling, and evalua-

tions. Piezometers were installed and readings began in October 1980. As of June 1981 all readings taken at the project site indicated no positive increment in groundwater regime. Sequential and referenced photographs for this site are included as Exhibit No. II. Site location maps, plans, reference points, sections, profiles, and details are included as Exhibit No. I.

B. Evaluation of Protection Performance.

1. General. All treatments have most probably reduced erosive losses of bank materials. Bank failure and soil removal mechanisms considered in the development of design concepts for treatment alternatives which are not generally significant are: wind-induced and navigation-generated waves and weather conditions (rainfall impact and runoff, and freeze and thaw cycles)

2. Scheme A. The tire mat has retained sediments. Limited areas of piping have occurred in the upper bank.

3. Scheme B. The soil cement filled bags and bulk geometries evidence no displacement or deterioration. The upper bank has continued to erode by piping.

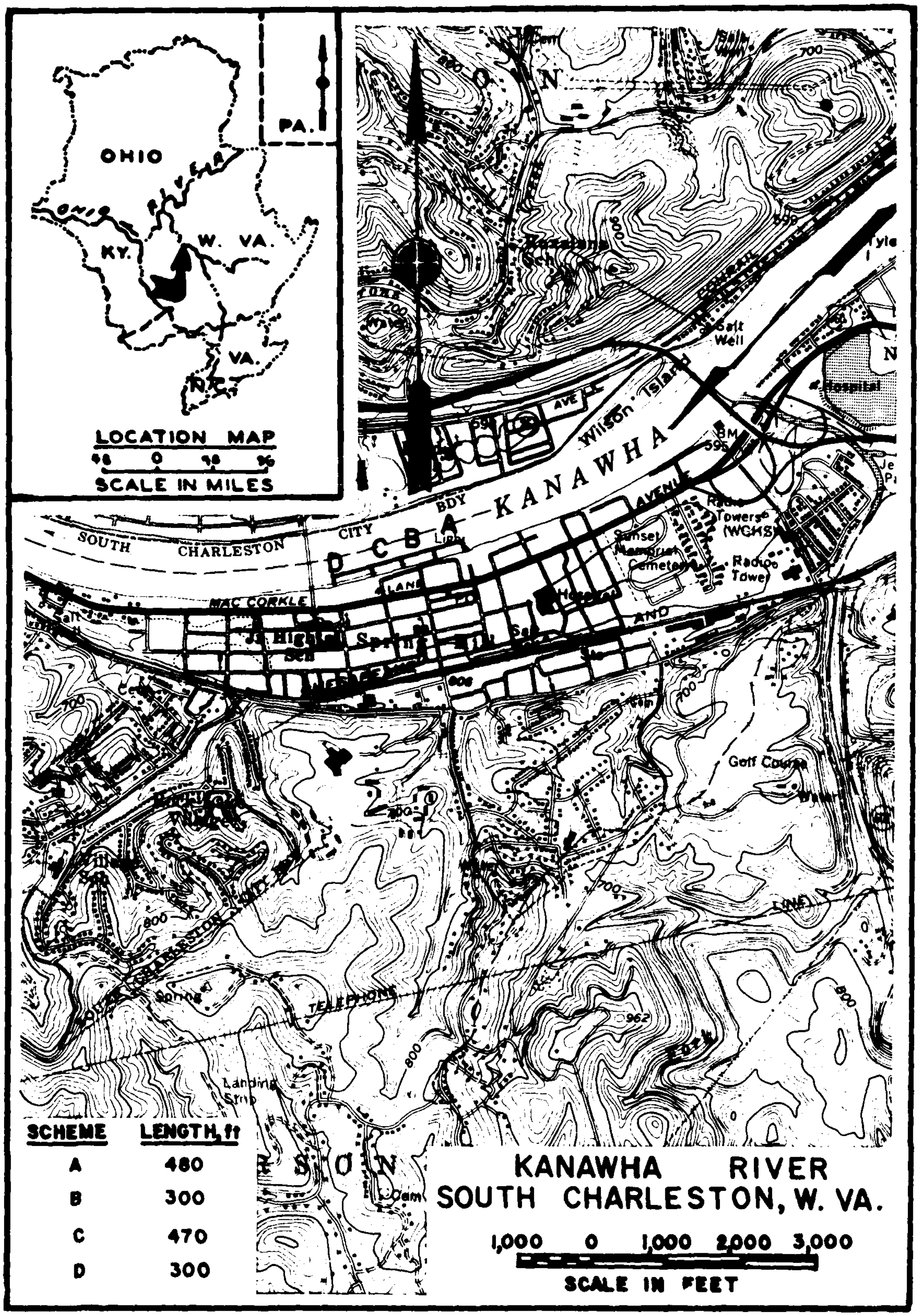
4. Scheme C. The floating tire breakwater continues to collect drift and is often partially submerged. The upper bank contains limited areas of erosion as a result of seasonally persistent seepages.

5. Scheme D. The graded stone does not evidence significant displacement or deterioration. The ungraded upper bank has continued to erode by piping, related slabbing, and drawdown initiated slumpages.

C. Rehabilitation. No rehabilitation work has been performed.

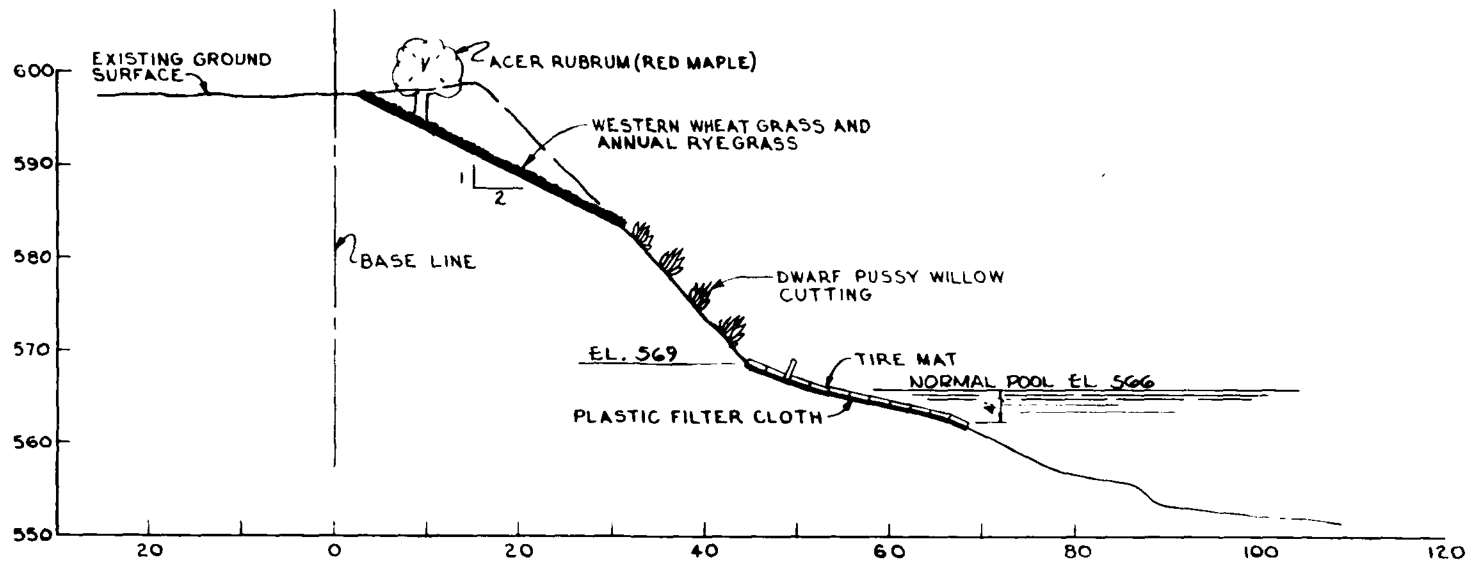
D. Summary of Findings. The upstream river limit of these treatments consists of a dumped riprap section approximate to Thomas Street. This treatment seems to be performing acceptably and is a rather standard method of addressing bank failure and erosion for this reach of the Kanawha River. The adjacent downstream treatment, Section A, consists of a tire mat, where the upper bank evidences some piping and storm and wastewater discharge-related erosion. The mat is a location of fine

sand alluvium accumulations. The adjacent treatment, Section B, consists of soil-cement berms and soil-cement filled bags. This treatment is not distressed and has resulted in a net reduction in bank failure and erosion. The floating tire breakwater treatment, Section C, is ineffectual in addressing wave attacks, is a debris collector, and is largely submerged. It is recommended that this treatment be removed, that the relative insignificance of wave-related erosion be acknowledged. Random dumping of concrete and other inert debris by the City of South Charleston is considered as an acceptable bank failure and erosion treatment for this test section. At the most downstream treatment, Section D, placement of quarry run 2-1/2 to 5 inch size waste rock has proved to be rather effective in reducing bank failure erosion. The upper reach of this bank section continues to fail as was anticipated in the development of designs and affecting of constructed treatment and top of bank retreat results.



SCHEME	LENGTH, ft
A	480
B	300
C	470
D	300

EXHIBIT I-1



SCHEME A

KANAWHA RIVER
SOUTH CHARLESTON, W. VA.

EXHIBIT I-2

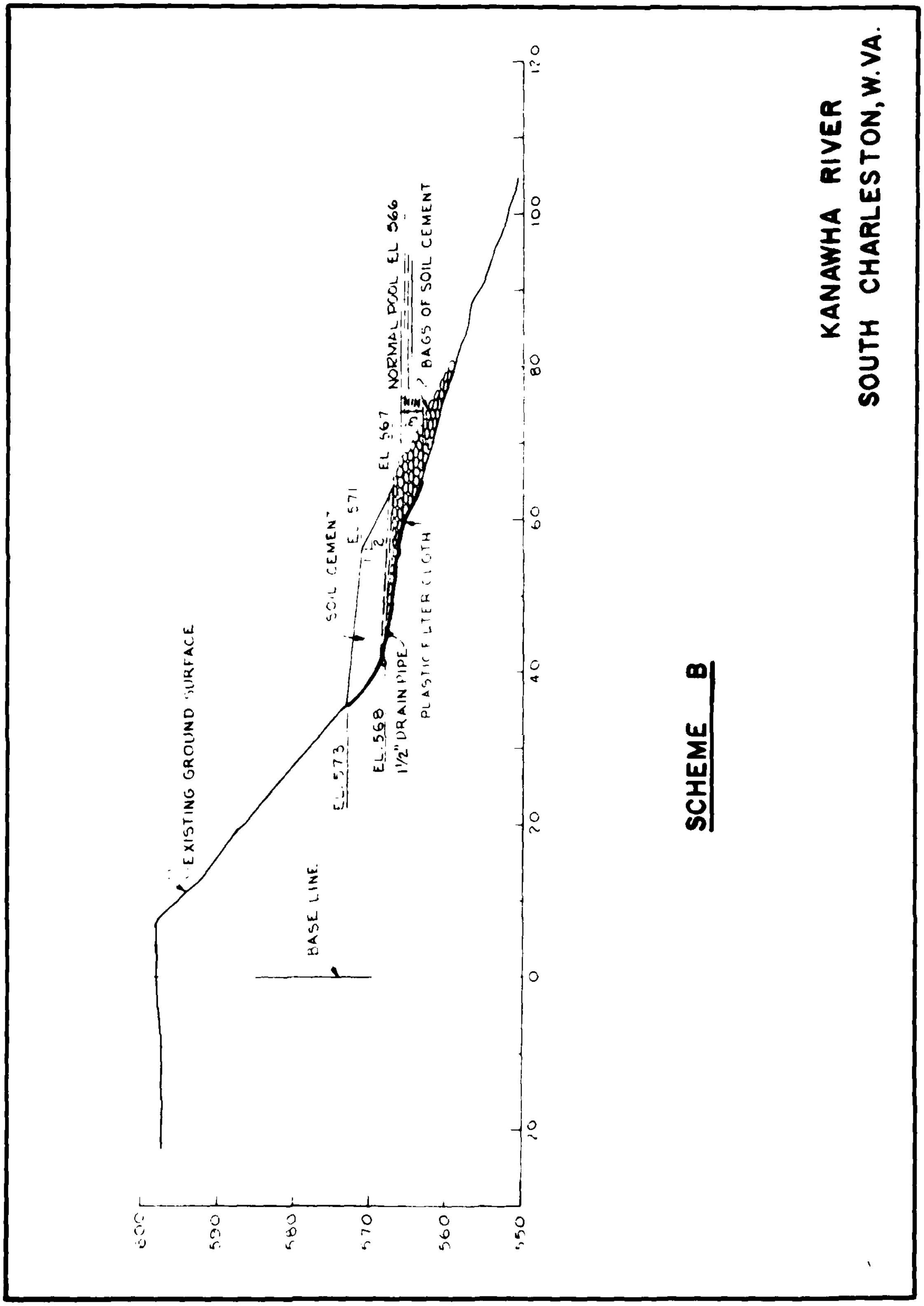
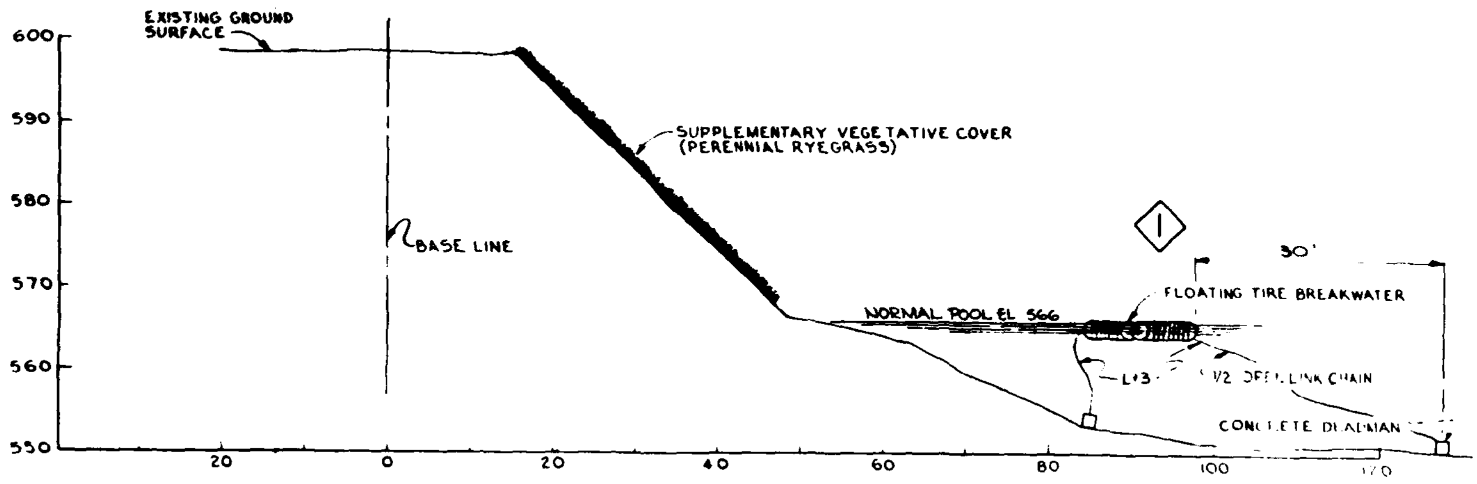

KANAWHA RIVER
SOUTH CHARLESTON, W.VA.
SCHEME B
EXISTING GROUND SURFACE
BASE LINE
SOIL CEMENT
EL 573
EL 568
1½" DRAIN PIPE
PLASTIC FILTER CLOTH
EL 571
EL 567
NORMAL POOL EL 566
BAGS OF SOIL CEMENT
600
590
580
570
560
550
0
20
40
60
80
100
120

EXHIBIT I-3



SCHEME C

KANAWHA RIVER
SOUTH CHARLESTON, W. VA.

EXHIBIT I-4

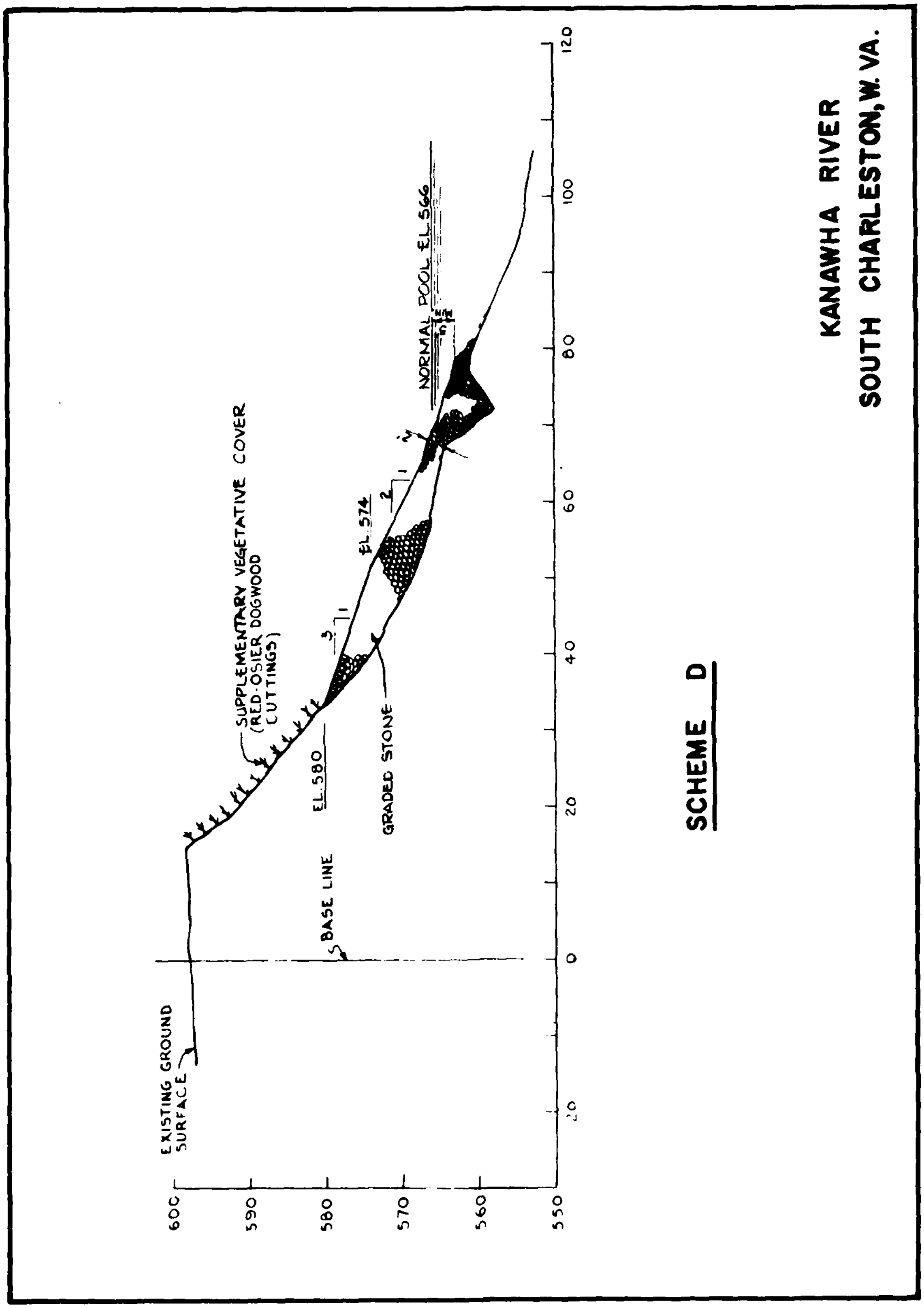

KANAWHA RIVER
SOUTH CHARLESTON, W. VA.
SCHEME D
SUPPLEMENTARY VEGETATIVE COVER
(RED-OSIER DOGWOOD
CUTTINGS)
NORMAL POOL EL. 566
EL. 574
EL. 580
GRADED STONE
BASE LINE
EXISTING GROUND
SURFACE
600
590
580
570
560
550
20
0
20
40
60
80
100
120

EXHIBIT I-5

COST SUMMARY
South Charleston, W. Va., Demonstration Project

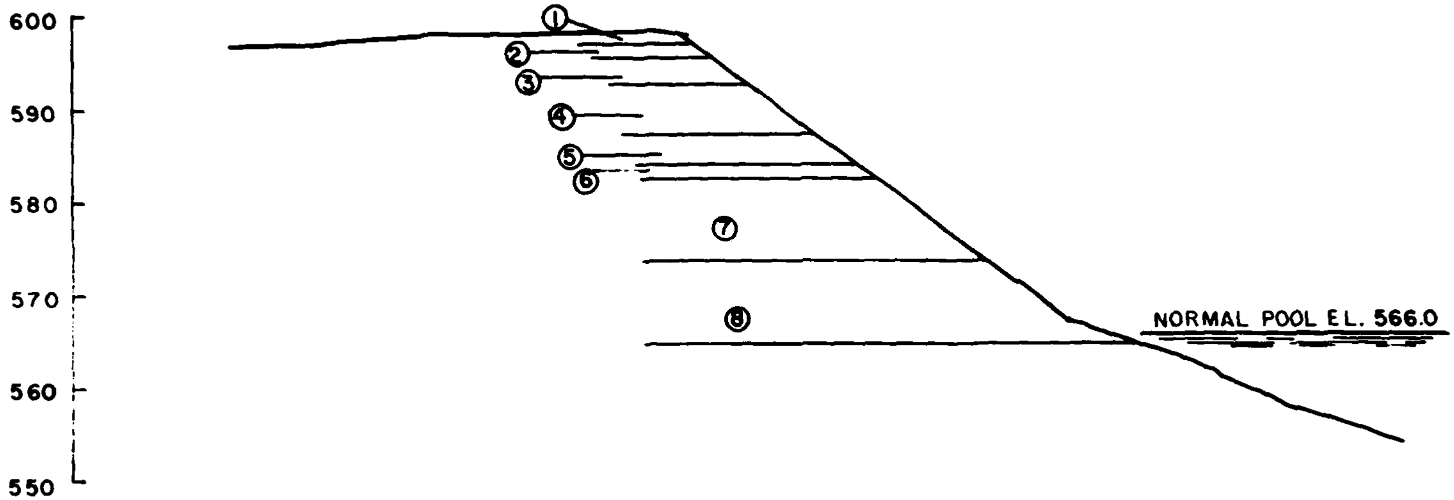
ITEM	TOTAL COST	COMMENTS
Construction	$ 468,885	Construction work done by lump sum contract
Engineering & Design	55,330	
Supervision & Administration	20,340	
Monitoring	900	
Reconstruction	0	
TOTAL	$ 545,455	

CONSTRUCTION COST FOR EACH TREAMTMENT

Type of Protection		Cost per Lineal foot	Cost per Square foot
Scheme A:	tire mat	$ 270.93	$ 4.17
Scheme B:	soil cement	315.63	7.01
Scheme C:	tire breakwater	218.62	2.19
Scheme D:	stone	471.33	6.73

These cost include all preparation of slopes and installation

EXHIBIT I-6



① SILTY SANDY CLAY, mst., pl., /rts., .2' top soil

② SANDY SILT, br., mst. – damp, non pl.

③ SILTY SANDY CLAY, br., pl., mst.

④ SILTY SANDY CLAY, gr. – gr. br., mot., pl., mst./sa. se.

⑤ SILTY CLAYEY SAND, gr., non pl., mst. – wet, f. g.

⑥ SILTY SAND, gr., non pl., wet, f. g./iron sta.

⑦ SILTY SAND, br., non pl., wet, compact – loose/sa se., f.g., ss. frag.

⑧ SILTY SAND, br., non pl., wet, f.g., gr. mud at 567.5 and 566.0

GEOTECHNICAL CROSS SECTION
SOUTH CHARLESTON, W. VA.

EXHIBIT I-7 (SHEET 1 OF 2)

ABBREVIATIONS

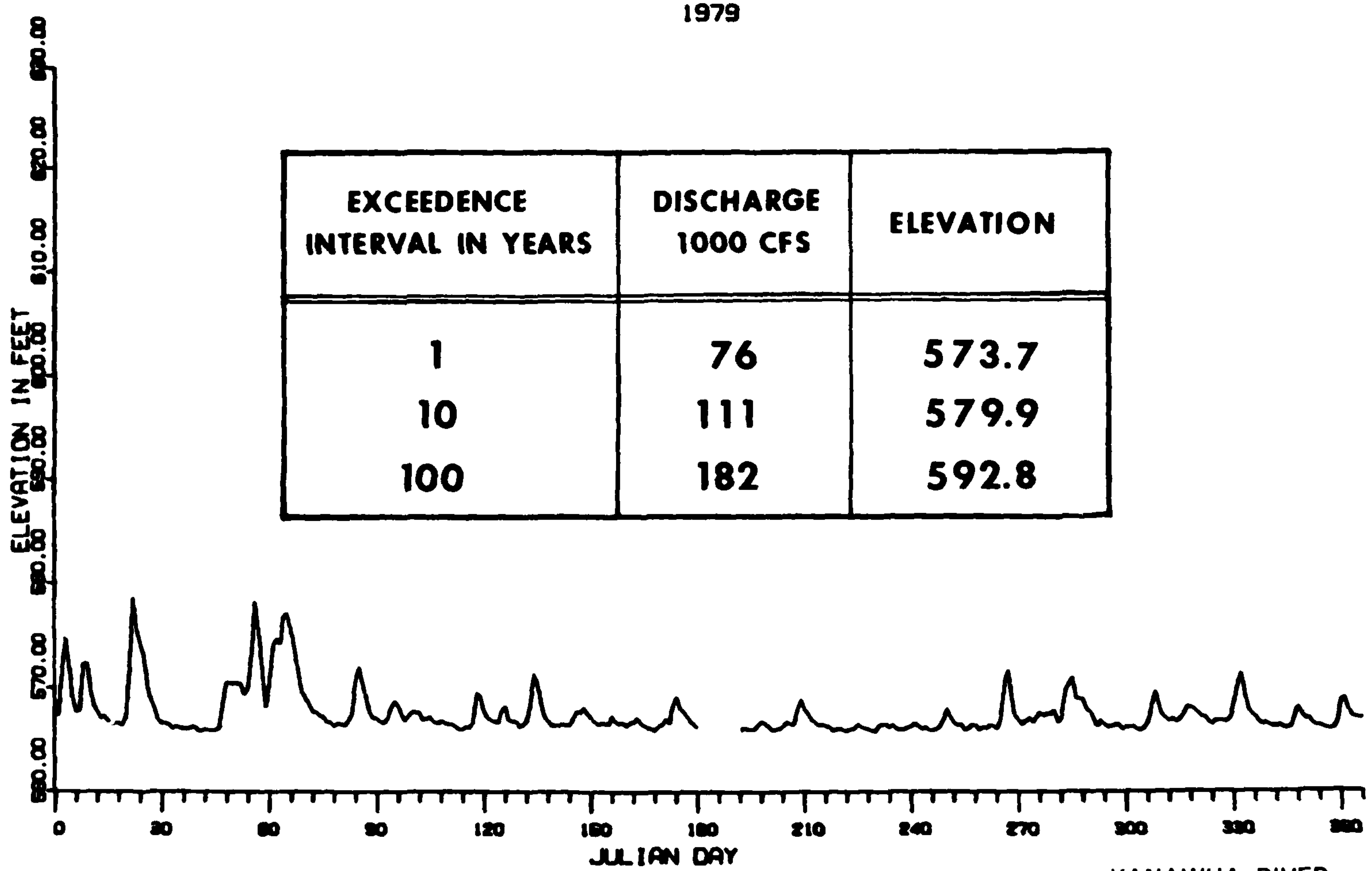
a.	angle
alt.	alternat(e)(ly)(ing)
amt.	amount
ang.	angular
approx.	approximate(ly)
ar.	argillaceous
aren.	arenaceous
asp.	asphaltic
b.	bone
ba.	banded(ing)
bd.	bed(ded)(ing)
bdr.	bedrock
bf.	buff
bk.	black
bky.	blocky
bkn.	broken
bl.	blue
bou.	boulder(s)
bre.	breccia(ted)
br.	brown
c.	coarse
ca.	calcareous
carb.	carbonaceous
cav.	cavern, cavity
cbl.	cobble(y)
ch.	chert
cl.	clayey
cle.	clean
coa.	coated(ing)
comp.	compact
conc.	concretion
cong.	conglomeratic
cont.	contains
cr.	crushed
crm.	crumbly
cst.	crystal(line)
cem.	cement(ed)
di.	dirty
dia.	diameter
diag.	diagonal
dis.	disintegrated
disc.	discontinuous
diss.	disseminated
dk.	dark
dn.	dense
dmp.	damp
ext.	extremely
f.	fine
fer.	ferruginous
fis.	fissile
fil.	filled(ing)
fos.	fossil(iferous)
frac.	fracture(d)
frags.	fragment(s)(al)
fri.	friable
f.w.	free water
g.	grain(ed)
gen.	generally
gn.	green
gr.	gray
gra.	gravelly
grad.	grading(ed)
G.W.	ground water
h.	hard
ha.	high angle
hi.	high(ly)(er)
hor.	horizontal(ly)
inc.	inclusions
incr.	increasing(ly)
inla.	interlaminated
intbd.	interbedded
irr.	irregular
jt.	joint(ed)
l.	little
la.	low angle
las.	laminat(ions)(ed)
lay.	layer(s)
le.	lean
lea.	leached
len.	lense(s)
lg.	large
lt.	light
m.	moderate(ly)
ma.	many
mas.	massive(ly)
mat.	material
mic.	micaceous
min.	mineralized
mos.	mostly
mot.	mottled
mst.	moist
mtx.	matrix
n.	near
nod.	nodule(s)
num.	numerous
o.	open
occ.	occasional(ly)
occu.	occurring
org.	organic
pa.	parting(s)
part.	particle(s)
pl.	plastic
peb.	pebble(s)
pk.	pink
pkt.	pocket(s)
pit.	pit(ted)
pn.	plane(s)
po.	porous
pt.	part(ly)
pyr.	pyrite(ic)
q.	quartzitic
r.	red
ro.	rock(s)
rot.	rotten(ed)
rou.	round(ed)
rt.	root(s)(let)
s.	soft
sa.	sandy
sat.	saturated
scat.	scattered
se.	seams
sev.	severely
sevr.	several
sh.	shaly
sil.	siliceous
sl.	silty
sli.	slight(ly)
slk.	slickensided
sm.	small
so.	some
sol.	solution
sta.	stain(ed)
stf.	stiff
stks.	streak(s)
str.	stringer(s)
sty.	stylolite(ic)
t.	thin
tho.	throughout
tk.	thick
tr.	trace
v.	variably
va.	variegated
ve.	very
veg.	vegetation
ver.	vertical(ly)
vu.	vuggy
w.	water
/	with
w.c.	water content
wd.	weathered
whi.	white
x-bd.	cross bedded(ing)
y.	yellow
zo.	zone

GEOTECHNICAL CROSS SECTION

EXHIBIT I-7 (SHEET 2 OF 2)

1979

EXCEEDENCE INTERVAL IN YEARS	DISCHARGE 1000 CFS	ELEVATION
1	76	573.7
10	111	579.9
100	182	592.8



KANAWHA RIVER
SOUTH CHARLESTON, W. VA.
STAGE HYDROGRAPHS

EXHIBIT I-8 (SHEET 1 OF 2)

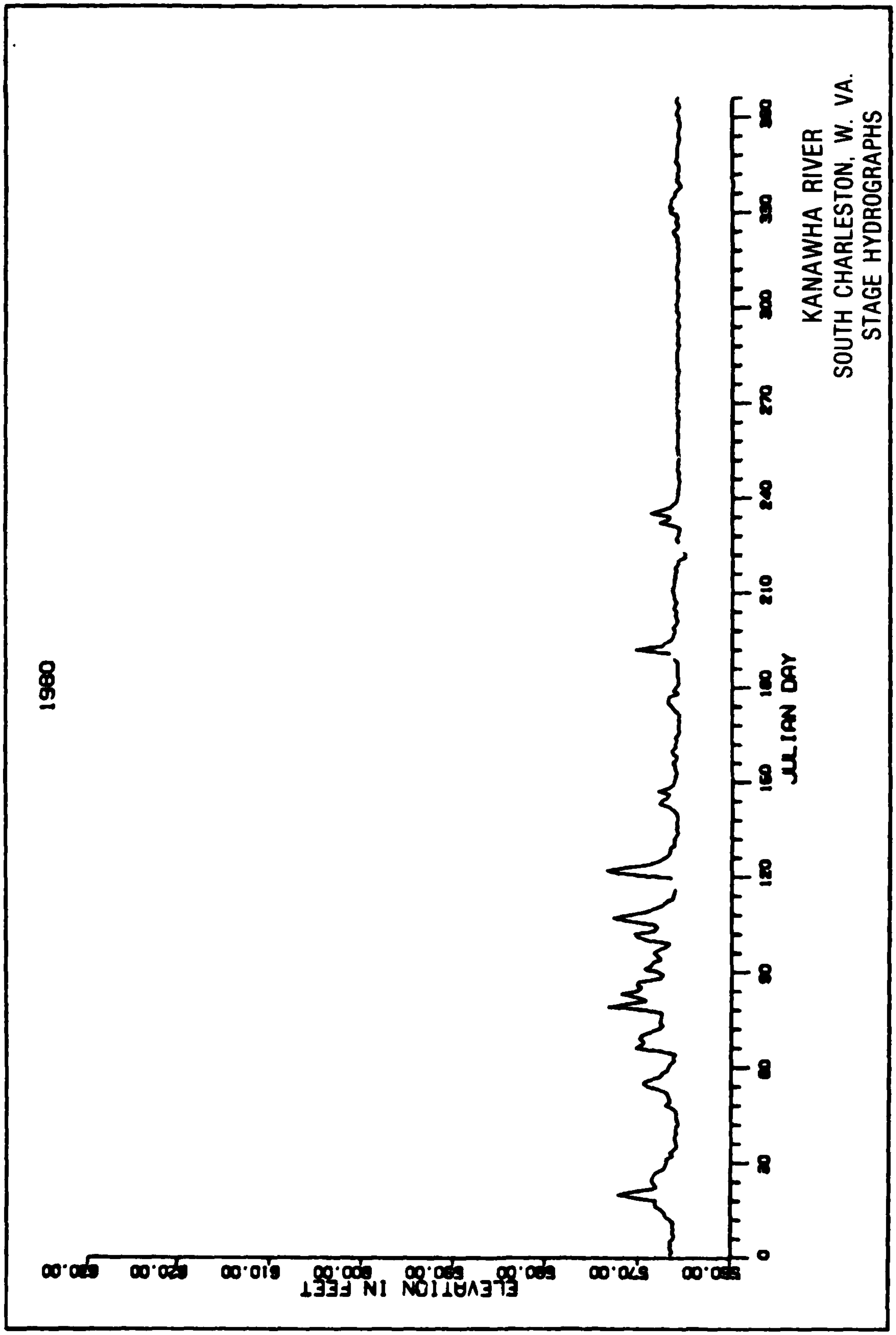


EXHIBIT I-8 (SHEET 2 OF 2)

SCHEME A: LOOKING DOWNSTREAM BEFORE CONSTRUCTION

SCHEME A: LOOKING UPSTREAM DURING CONSTRUCTION

EXHIBIT II-1

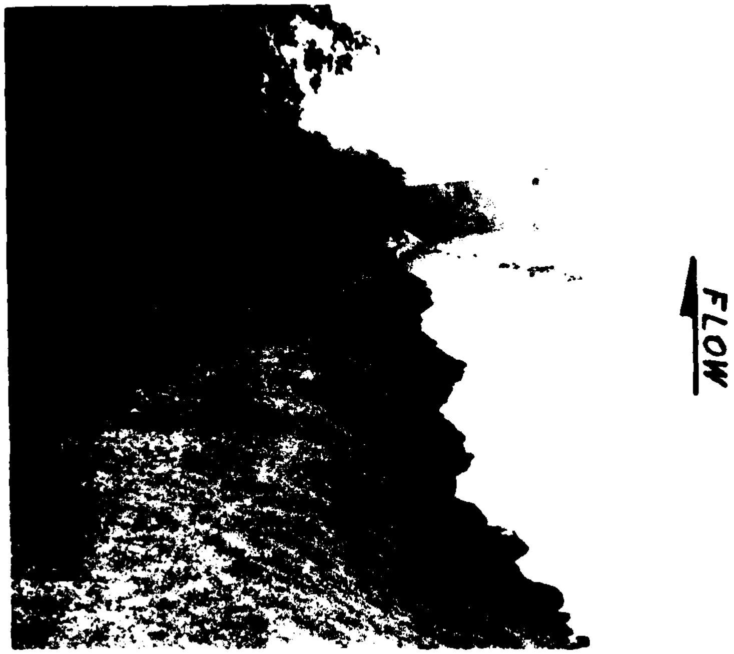
SCHEME A: LOOKING UPSTREAM AFTER CONSTRUCTION

EXHIBIT II-2

SCHEME B: LOOKING UPSTREAM BEFORE CONSTRUCTION

SCHEME B: LOOKING UPSTREAM DURING CONSTRUCTION

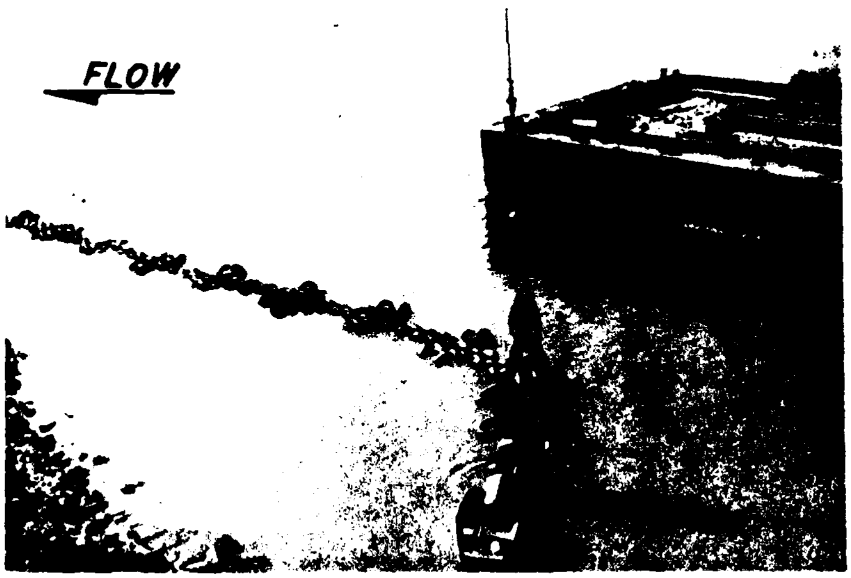
EXHIBIT II-3



SCHEME B: LOOKING DOWNSTREAM AFTER CONSTRUCTION

EXHIBIT II-4

SCHEME C: LOOKING UPSTREAM BEFORE CONSTRUCTION



SCHEME C: LOOKING DOWNSTREAM DURING CONSTRUCTION

EXHIBIT II-5

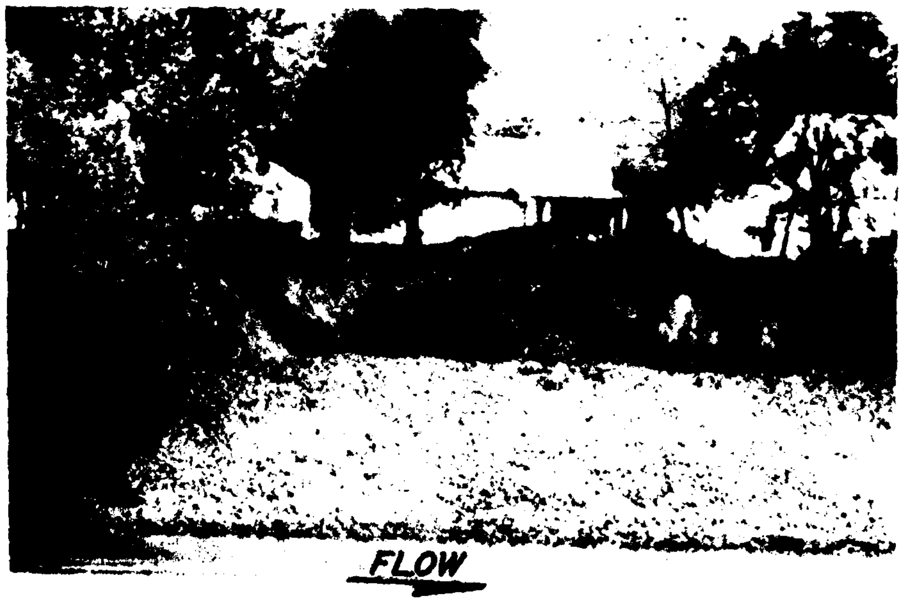
SCHEME C: LOOKING DOWNSTREAM AFTER CONSTRUCTION

EXHIBIT II-6

SCHEME D: LOOKING UPSTREAM BEFORE CONSTRUCTION

SCHEME D: LOOKING DOWNSTREAM DURING CONSTRUCTION

EXHIBIT II-7



SCHEME D: LOOKING DOWNSTREAM AFTER CONSTRUCTION

EXHIBIT II-8

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report H-77-9	2. GOVT ACCESSION NO. AD-A121 138	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) LITERATURE SURVEY AND PRELIMINARY EVALUATION OF STREAMBANK PROTECTION METHODS		5. TYPE OF REPORT & PERIOD COVERED Final report
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Malcolm P. Keown Noel R. Oswalt Edward B. Perry Elba A. Dardeau, Jr.		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Hydraulics Laboratory, Mobility and Environmental Systems Laboratory, Soils and Pavements Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS Work Unit 02
11. CONTROLLING OFFICE NAME AND ADDRESS Office, Chief of Engineers, U. S. Army Washington, D. C. 20315		12. REPORT DATE May 1977
		13. NUMBER OF PAGES 262
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Authorized for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Bank erosion
Bank protection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A preliminary study of streambank erosion control was conducted with the major emphasis on an extensive literature survey of known streambank protection methods. In conjunction with the survey, preliminary investigations were conducted to identify the mechanisms that contribute to streambank erosion and to evaluate the effectiveness of the most widely used streambank protection methods. The results of the literature survey and the two preliminary investigations are presented herein.

(Continued)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued).

The text of the "Streambank Erosion Control Evaluation and Demonstration Act of 1974" is presented in Appendix A. A list of commercial concerns that market streambank protection products is provided in Appendix B. Appendix C contains a glossary of streambank protection terminology. A detailed bibliography resulting from the literature survey is provided in Appendix D, and a listing of selected bibliographies related to streambank protection are provided in Appendix E.